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TRANSIT COOPERATIVE RESEARCH PROGRAM

TCRP REPORT 155

Track Design Handbook for Light Rail Transit

Second Edition

Parsons Brinckerhoff, Inc.
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TRANSIT COOPERATIVE RESEARCH PROGRAM

The nation's growth and the need to meet mobility, environmental, and energy objectives place demands on public transit systems. Current systems, some of which are old and in need of upgrading, must expand service area, increase service frequency, and improve efficiency to serve these demands. Research is necessary to solve operating problems, to adapt appropriate new technologies from other industries, and to introduce innovations into the transit industry. The Transit Cooperative Research Program (TCRP) serves as one of the principal means by which the transit industry can develop innovative near-term solutions to meet demands placed on it.

The need for TCRP was originally identified in *TRB Special Report 213—Research for Public Transit: New Directions,* published in 1987 and based on a study sponsored by the Urban Mass Transportation Administration—now the Federal Transit Administration (FTA). A report by the American Public Transportation Association (APTA), Transportation 2000, also recognized the need for local, problem-solving research. TCRP, modeled after the longstanding and successful National Cooperative Highway Research Program, undertakes research and other technical activities in response to the needs of transit service providers. The scope of TCRP includes a variety of transit research fields including planning, service configuration, equipment, facilities, operations, human resources, maintenance, policy, and administrative practices.

TCRP was established under FTA sponsorship in July 1992. Proposed by the U.S. Department of Transportation, TCRP was authorized as part of the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA). On May 13, 1992, a memorandum agreement outlining TCRP operating procedures was executed by the three cooperating organizations: FTA, the National Academies, acting through the Transportation Research Board (TRB); and the Transit Development Corporation, Inc. (TDC), a nonprofit educational and research organization established by APTA. TDC is responsible for forming the independent governing board, designated as the TCRP Oversight and Project Selection (TOPS) Committee.

Research problem statements for TCRP are solicited periodically but may be submitted to TRB by anyone at any time. It is the responsibility of the TOPS Committee to formulate the research program by identifying the highest priority projects. As part of the evaluation, the TOPS Committee defines funding levels and expected products.

Once selected, each project is assigned to an expert panel, appointed by the Transportation Research Board. The panels prepare project statements (requests for proposals), select contractors, and provide technical guidance and counsel throughout the life of the project. The process for developing research problem statements and selecting research agencies has been used by TRB in managing cooperative research programs since 1962. As in other TRB activities, TCRP project panels serve voluntarily without compensation.

Because research cannot have the desired impact if products fail to reach the intended audience, special emphasis is placed on disseminating TCRP results to the intended end users of the research: transit agencies, service providers, and suppliers. TRB provides a series of research reports, syntheses of transit practice, and other supporting material developed by TCRP research. APTA will arrange for workshops, training aids, field visits, and other activities to ensure that results are implemented by urban and rural transit industry practitioners.

The TCRP provides a forum where transit agencies can cooperatively address common operational problems. The TCRP results support and complement other ongoing transit research and training programs.

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Chapter 1—General Introduction

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CHAPTER 1—GENERAL INTRODUCTION

1.1 INTRODUCTION

The purpose of this Handbook is to provide those responsible for the design, procurement, construction, maintenance, and operation of light rail transit (LRT) systems an up-to-date guide for the design of light rail track, based on an understanding of the relationship of light rail track and other transit system components. While this Handbook's title implies that it pertains only to light rail transit, individual principles discussed herein are applicable to a wide spectrum of railway operations ranging from low-speed streetcars operating in city streets up through metro rail and heavy rail transit lines in exclusive grade separated guideways. Some basic principles are universal, and designers of freight and passenger railroad systems will, upon perusal of the Handbook, likely also find chapters and articles of universal interest. The contents of the Handbook were compiled as a result of an investigation of light rail transit systems, a review of literature pertaining to transit and railroad standards and methods, and personal hands-on experience of the authors. Current research also has been a source of valuable data.

1.1.1 Background

This second edition of the *Track Design Handbook for Light Rail Transit* builds upon the first edition, which is also known as *TCRP Report 57*. *TCRP Report 57*, published in 2000, was the culmination of the TCRP Project D-6, which was initiated in 1995. TCRP Project D-6 came about because there was seemingly no consistency in the track design used on those North American light rail transit projects that had been initiated in the 1980s and early 1990s. While much research had been conducted in an effort to understand the mechanisms involved in track-rail vehicle interaction and its impact on track design, no widely accepted guidelines existed to specifically aid in the design and maintenance of light rail transit track. Other than the recommended practices of what was then called the American Railway Engineering Association (AREA), there was no up-to-date and commonly accepted resource of track design information to which a North American light rail transit designer could refer. Since AREA was primarily focused on freight railroads and since information on possibly more applicable design practices overseas was difficult to obtain and often unavailable in the English language, many light rail transit projects were designed using a hodgepodge of criteria, drawn from widely disparate sources. Light rail transit designers had little choice other than to rely on practices developed primarily for heavy rail transit and railroad freight operations that are not necessarily well suited for light rail systems. The result was design criteria that were often internally inconsistent. Moreover, many of those projects, once they had been built, had appreciable maintenance issues due to fundamental inconsistencies between their track designs and the vehicles that were using them.

TCRP Report 57 altered the field by providing a single source of information, and it was immediately accepted as an authoritative resource. It is upon that foundation that this Second Edition is built.

1.1.2 Purpose and Goals of the Handbook

The purpose of this Handbook is to offer a range of design guidelines, not to set a universal standard for an industry that operates in a wide range of environments. The Handbook furnishes

the reader with current practical guidelines and procedures for the design of the various types of light rail track—including ballasted, direct fixation, and embedded track systems—and offers choices concerning the many issues that must be resolved during the design process. It discusses the interrelationships among the various disciplines associated with light rail transit engineering—structures, traction power, stray current control, noise and vibration control, signaling, and electric traction power. It also describes the impacts of these other disciplines on trackwork and offers the track designer insights into the requisite coordination efforts between all disciplines.

A key focus of the Handbook is to differentiate between light rail transit track and those similar, but subtly different, track systems used for freight, commuter, and heavy rail transit operations. These differences present challenges both to light rail track designers and to the designers and manufacturers of light rail vehicles.

There will always be some indeterminacy in the engineering mechanics of light rail transit trackwork because the system is dynamic and functions in the real world. LRT track is subject not only to the vagaries of wear and tear but also to the realities of funding for maintenance in a highly politicized environment. Therefore, while perfection can and should be strived for—particularly during initial construction, when funding is easier to obtain—it can never be achieved. It should also be noted that trackwork for all types of railways traces its heritage back to animal-powered colliery tramways of the late 18th century. The fundamental design principles that were then selected for those then-new “rail roads” constrain what is practical to achieve now. Some problems of the rail/wheel interface will likely be forever intractable because of decisions made over two centuries ago. Hence, maintenance-free track for a light rail system is not plausible.

1.1.3 The Handbook User

The user of the Handbook assumes all risks and responsibilities for selection, design, and construction to the guidelines recommended herein. No warranties are provided to the user, either expressed or implied. The data and discussions presented herein are for informational purposes only.

The reader is assumed to be a degreed civil engineer or similarly qualified individual who is generally familiar with trackwork terminology and experienced in the application of guideline information to design. For that reason, a glossary of terms that would be familiar to a trackwork engineer has not been included herein. Definitions of common trackwork terms are included in the *Manual for Railway Engineering*, published by the American Railway Engineering & Maintenance-of-Way Association (AREMA). Terms that are unique to light rail transit are defined within the text of the Handbook as they are introduced.

Design and construction of light rail transit projects is a multidisciplinary effort. The reader is presumed to be the person on the project who is responsible not only for the design and specification of trackwork hardware, but also for the design of the track alignment. However, LRT projects are not only multidisciplinary, they are interdisciplinary. It is not possible for any one individual to work separately from the other disciplines.

In the case of the track alignment engineer, he or she will obviously need to work closely with other civil engineers on the project who are responsible for earthworks, drainage, and roadway work and the structural engineers responsible for bridges, walls, and other guideway structures. Less obvious, but just as important, is the need to coordinate with the following other team partners:

- The operations planners, so the track alignment is supportive of the operating plan. This is not only with respect to where the tracks go, but also meeting the operating speed objectives and providing crossover tracks and turnback/pocket tracks at requisite locations.
- The designers of the overhead contact system (OCS), so as to be certain that a suitable OCS alignment can be created above the track alignment.
- The train control system designers, so the track speeds are synchronized with the maximum speeds the signal system can permit.
- The vehicle engineers for vital information about the all-critical rail-to-wheel interface as well as any other restrictions, such as minimum possible curve radius or maximum gradient that the vehicle might impose on the design.
- The station architects and site planners when setting the locations of the station platforms.
- The traffic engineers, so that interface locations between the LRT tracks and public roadways are configured in a manner that facilitates the smooth and safe operation of rail, rubber-tired, and pedestrian traffic.
- The yard and shop design team so that a site's typically constrained real estate is used in an efficient manner with due recognition of the fact that track geometry is usually the least flexible component of the overall yard design.

In the user's role as trackwork designer, interfaces will again be required with multiple other disciplines, including most of the list above. Trackwork interfaces will include the traction power engineers for negative return connections to the track, structural engineer for interaction between the track and the bridges that support it, signal engineers for train control attachments to the track such as switch machines and insulated joints, highway engineers for the configuration of roads that are either crossed or occupied by the light rail tracks, vehicle engineers for coordination of the crucially important rail/wheel interface, and a host of others. The track engineer needs to understand the role each of those other parties has in the project, the basic principles associated with the facilities or systems that they design, how those details relate to the track, and be able to ask intelligent questions when appropriate. This Handbook is designed to give the track designer the background necessary to do just that.

From the above, clearly the track alignment/trackwork engineer occupies a central position on a light rail transit project. Indeed, the track engineer probably interfaces with more people on the project team than anybody except project management! It's a crucial and exciting role! Enjoy it!

1.2 WHAT IS LIGHT RAIL?

1.2.1 Background

Light rail transit evolved from streetcar technology. Electric streetcars dominated urban transit in just about every significant American city up through World War II. But once the war was over, "old-fashioned" trolley lines were converted to bus operation in droves, all in the name of "modernization." By 1965, only a handful of legacy streetcar systems still survived.

The genesis of the terminology "light rail transit" in the United States dates to the late 1960s when planning efforts were underway at what was then called the Urban Mass Transit Administration (today's Federal Transit Administration) to procure new vehicles for legacy trolley lines in Boston and San Francisco. The principals working on that program recognized that, because of the wholesale abandonment of streetcar lines in the previous two decades, the words "streetcar" and "trolley" had stigmas with likely negative political consequences for the program. Therefore, the term "light rail vehicle" was coined, borrowing from British vernacular.

1.2.2 Light Rail Defined

Tracks for light rail transit are generally constructed with the same types of materials used to construct "heavy rail," "commuter rail," and railroad freight systems. Also, light rail vehicles may be as massive as transit cars on heavy rail systems. Consequently, the term "light rail" is somewhat of an oxymoron and often misunderstood. Therefore, for the purposes of this book, it is appropriate to define light rail transit.

The American Public Transportation Association (APTA) defines light rail transit as

> *An electric railway system characterized by its ability to operate single or multiple car consists along exclusive rights-of-way at ground level, on aerial structures, in subways or in streets, able to board and discharge passengers at station platforms or at street, track, or car-floor level and normally powered by overhead electrical wires.*

To expand that definition:

- Light rail is a system of electrically propelled passenger vehicles with steel wheels that are propelled along a track constructed with steel rails.
- Propulsion power is drawn from an overhead distribution wire by means of a pantograph or other current collector and returned to the electrical substations through the rails.
- The tracks and vehicles must be capable of sharing the streets with rubber-tired vehicular traffic and pedestrians. The track system may also be constructed within exclusive rights-of-way.
- Vehicles are capable of negotiating curves as sharp as 25 meters [82 feet] and sometimes even sharper, in order to traverse city streets.
- Vehicles are not constructed to structural criteria (primarily crashworthiness or "buff strength") needed to share the track with much heavier railroad commuter and freight equipment.

1.2.3 Light Rail as a Spectrum

While, as noted above, the Handbook is applicable to railway track engineering for a wide spectrum of railway systems, its principal focus is light rail transit. LRT itself is a broad spectrum and ranges from single unit streetcars running in mixed traffic within city streets at speeds as slow as 25 mph [40 km/h] and even lower up through multiple car trains running on a totally exclusive guideway at speeds of 60 mph [100 km/h] or faster. The streetcar lines in New Orleans are representative of the lower end of this spectrum while the Metrolink system in St. Louis is a good example of the upper end. In much of Europe, these two extremes are often called "trams" and "metros." In Germany, the terms "strassenbahn" ("street railway") and "stadtbahn" ("city railway") are commonly used.

The focus of the first edition of this Handbook was more toward the stadtbahn end of the LRT continuum, since they were the prototype for nearly all North American LRT projects during the 1980s and 1990s. However, because of the resurgence of North American streetcar operations during the first decade of the 21st century, it is appropriate for this second edition of the Handbook to provide additional information on the track alignment and trackwork for strassenbahn-type operations.

It is important to note how, along any given light rail transit line, one might reasonably include guideway and track elements that are very much like a strassenbahn while a short distance away the route's character might radically change into that of a stadtbahn. LRT is a continuum and, within the framework of the operating requirements of a given project, the LRT track designer can incorporate appropriate elements from each of the mode's extreme characteristics plus just about anything in between.

Light rail lines are fairly distinct from metro rail systems (often called "heavy rail"). The latter are always entirely in exclusive rights-of-way, are usually designed to handle long trains of vehicles (6 to 10 cars per train is common) and have a relatively high absolute minimum operating speed along the revenue route (usually 45 mph [72 km/h] or higher). By contrast, LRT trains can operate in shared rights-of-way, very seldom exceed three cars per train, and speeds as low as 10 mph [16 km/h] are tolerated in revenue service track. These differences usually mean that LRT can be constructed at far lower cost than metro rail transit, although the passenger throughput capacity of the latter is also much higher.

If there is any one single characteristic that defines "light rail," it is likely the ability of the vehicle to operate in mixed traffic in the street when necessary. This draws a line between the St. Louis example above and a light metro rail operation such as SEPTA's Norristown high speed line. The operational characteristics of each route are virtually the same, but only the St. Louis vehicle could actually operate in the street if necessary. It is a very fine distinction, and, while purists may quibble with some of the finer points of this definition, it will suffice for the purposes of this Handbook.

Several rail transit projects have utilized diesel-powered light railcars (also known as "diesel mechanical units" or "DMUs"), which do not meet FRA buff strength criteria. Except for the propulsion system, many of these vehicles and the guideways they run upon closely resemble the

stadtbahn end of the LRT spectrum. The second edition of the Handbook will not attempt to cover all of the nuances of the DMU mode; however much of the information contained in the Handbook will be directly applicable to professionals working on a DMU project.

Throughout this volume, the words “railroad” and “railway” will appear. By “railroad” the authors mean standard gauge rail operations that are part of the general system of railroad transportation. This includes freight railroads and passenger railroads (such as Amtrak and the commuter rail operations in many cities). The word “railway,” on the other hand, is intended as a broader term that includes all transportation operations that utilize a vehicle guidance system based on the use of flanged steel wheels riding upon steel rails.

1.2.4 Where the Rails and Wheels Meet the Road

Arguably, the two most important defining elements of trackwork for light rail systems are the construction of track in streets and the interface between the wheel of the light rail vehicles and the rails. Track in streets requires special consideration, especially with regard to the control of stray electrical current that could cause corrosion. These embedded tracks also need to provide a flangeway that is large enough for the wheels but does not pose a hazard to other users of the street.

Light rail vehicle wheels do not necessarily match those used in freight railroad service. Wheel diameters are usually much smaller, and the wheel tread is often much narrower. Light rail wheel flanges are often shorter and have a radically different contour than railroad wheels. These variations require special care in track design, especially in the design of special trackwork such as switches and frogs. The compatibility of the vehicle and track designs is a central issue in the development of a light rail system if both components are to perform to acceptable standards. These issues are discussed at length in this Handbook.

While light rail may need to share right-of-way (R/W) with pedestrians and vehicles, the designer should create an exclusive R/W for light rail tracks wherever possible. This will make operation more reliable and maintenance less expensive. Exclusive R/W can also simplify compliance with the Americans with Disabilities Act Accessibility Guidelines (ADAAG) and similar requirements in other countries.

1.2.5 The Regulatory Environment

Virtually every aspect of the operation and maintenance of railroads in the United States is closely regulated by the Federal Railroad Administration (FRA) of the U.S. Department of Transportation. However, very few rail transit operations are subject to any level of FRA oversight and regulation. In fact, as of 2010, the U.S. federal government does not exercise any direct oversight of rail transit operations. Instead, through 49 CFR, Part 659, *Rail Fixed Guideway Systems: State Safety Oversight*, the U.S. government delegates that responsibility to the states. Therefore, Handbook users must familiarize themselves with any applicable regulations in the state where the light rail transit line will be constructed and operate.

1.3 HANDBOOK ORGANIZATION

Chapter 1 (this chapter) provides general introductory information.

Chapter 2 elaborates on vehicle design and critical issues pertaining to track and vehicle interface. These topics include wheel/rail profiles, truck steering within restricted curves and primary and secondary suspension systems, and the effect of these parameters on track and operations.

Chapter 3 details issues related to light rail track geometry with particular attention to restrictions imposed by alignment characteristics, such as tight radius curvature, severe vertical curves, and steep profile grade lines.

Chapter 4 elaborates on the three basic types of track structures: ballasted, direct fixation, and embedded track. The chapter takes the designer through a series of selections pertaining to the track design. The chapter discusses track and wheel gauges, flangeways, rail types, guarded track (restraining rail), and track modulus and provides references to discussions on stray current, noise and vibration, and signal system and traction power requirements in other chapters.

Chapter 5 discusses various trackwork components and details.

Chapter 6 provides guidelines for the design and selection of various types and sizes of special trackwork. Included are details pertaining to switches, frogs, guard rails, crossings (diamonds), and associated items.

Chapter 7 recognizes that virtually all light rail transit systems require bridges or similar structures. Aerial structures are not uncommon. Chapter 7 provides a framework for determining the magnitude of forces generated due to differential thermal expansion between the rail (especially stationary continuous welded rail) and the structure. The analysis elaborates on structural restrictions, fastener elastomer displacement, fastening toe loads, friction and longitudinal restraint, and probable conditions at a rail break on the structure. The analysis includes the conditional forces generated by locating special trackwork on an aerial structure and methods of contending with them. The chapter also addresses the design issues of track slabs constructed on grade, particularly embedded track, since the design principles are distinctly different than ordinary roadway pavement.

Chapter 8 stems from the fact that light rail transit uses the running rail as a negative return in the traction power system and highlights the issues pertaining to stray current and discusses the need to electrically insulate the rail and thereby retard the potential for electrical leakage. Methodologies for establishing magnitude, identifying sources, and developing corrective measures are part of this chapter.

Chapter 9 introduces the designer to another environmental issue pertaining to light rail transit—noise and vibration. It explains wheel/rail noise and vibration and the fundamentals of acoustics. It also discusses mitigation procedures and treatments for tangent, curved, and special trackwork.

Chapter 10 highlights issues related to signals and related train control systems for light rail transit and discusses some of the interfacing issues and components that must be considered by a track designer.

Chapter 11 presents elements pertinent to traction power, including supply system and substations, the catenary distribution system, and the power return through the running rails. The chapter also discusses corrosion control measures to mitigate the effects of DC current to adjacent services.

Chapter 12 discusses issues related to the application of LRT into a street environment, particularly mixed traffic streetcar-style configurations.

Chapter 13 describes considerations the track designer should understand about how the project will actually be built. These include the "means and methods" of how the track constructor will actually perform the work and how the track construction activity will interact with the construction of other infrastructure and systems.

Chapter 14 describes the activities that will be necessary to maintain the track system in a state of good repair so that it can continue to meet the operational goals of the project. Emphasis is given to avoidance of details that either have high maintenance requirements or are difficult or impossible to routinely inspect and maintain.

An overall table of contents lists only the 14 chapter topics. Each chapter contains its own detailed table of contents; list of figures and tables; and, in some cases, a reference list. Pages are numbered by chapter (for example, 4-24 is page 24 in Chapter 4). Exhibits within each chapter are assigned a three-digit number indicating the chapter and article in which it appears. For example, Figure 7.3.4 would be the fourth exhibit in Article 7.3 of Chapter 7.

1.4 UNITS OF MEASUREMENT

The first edition of the *Track Design Handbook* (*TCRP Report 57*), published in 2000, utilized metric (SI) units as the primary system of measurement, with U.S. traditional units following [in brackets]. This was in keeping with federally mandated standards at the time TCRP Project D-6 was being performed. Since then, the legal mandate to transition to SI units of measurement has been repealed. The TCRP Project D-14 scope therefore required reversal of the pattern used in *TCRP Report 57*, i.e., this report uses U.S. traditional units first and SI units second [in brackets].

However, this revised protocol is reversed when the dimension being discussed is metric in origin, particularly in the case of products which are manufactured to SI dimensions. For example, all contemporary light rail vehicles are designed and constructed using SI units. In such cases, the metric version will be listed first, followed by a soft conversion into U.S. traditional units, e.g., 180 mm [7 inches]. In the rare event that an exact translation into U.S. traditional units is required, decimal inches or decimal feet may be employed, e.g., 180 mm [7.087 inches] instead of fractions.

Most unit conversions used in the Handbook are "soft" and therefore respect the practical tolerances implied by the primary dimension. For example, if a dimension is stated as

"approximately one foot," the conversion to SI is given as 300 millimeters or 30 centimeters rather than an inappropriately exact conversion to 304.8 millimeters.

Where formulae are used in the text, versions in both U.S. traditional units and SI units are provided. The authors have attempted to make the two versions of the formulae as consistent as possible so as to illustrate the process while also deriving answers that are generally consistent. In practice, there will be some divergence due to both the coarseness of the dimensional units in each system and the construction tolerances that are practical. For example, while a constructor might strive to place cross ties to 30-inch [762-mm] spacing, it is probable that as-built dimensions will vary plus or minus a half-inch [13 mm] from that dimension. This in no way invalidates the design because actual in-service loadings will always vary from the theoretical. In addition, it would be irrational for a constructor to attempt to place the cross ties precisely 762 millimeters apart or even 762 mm plus or minus 13 mm. If the project was being designed and constructed in SI units, it is likely that the actual specified cross tie spacing would be a value expressed in a unit that is both consistent with reasonably achievable tolerances and practical for field use—such as 75 cm plus or minus a centimeter.

1.5 THE ENDMARK

A common style feature in publishing is what is known as an "endmark." An endmark is a symbol, often with some relationship to the text that precedes it, that is placed at the end of an essay, chapter, or article. As its name implies, it means the reader has reached the end of the discussion. For this second edition of the *Track Design Handbook for Light Rail Transit*, the authors have selected as their endmark a simplified image of 140ER7B girder guard rail. That rail was a standard of the former American Transit Engineering Association (ATEA) and commonly used on North American streetcar lines up until circa-1960 when it became no longer available.

The ATEA itself disbanded in the decade following World War II as very nearly all cities in North America abandoned their trolley lines. Regrettably, streetcar trackwork professionals and their knowledge became widely dispersed. Fortunately, they left behind a notable comprehensive library of information on the design of trackwork for electric street railways—the ATEA's *Engineering Manual.* This endmark is a silent tribute to the now long-deceased authors of that volume, who in many ways knew far more about these topics than we can even hope to learn.

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Chapter 2—Light Rail Transit Vehicles

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CHAPTER 2—LIGHT RAIL TRANSIT VEHICLES

2.1 INTRODUCTION

The light rail transit vehicle ("light rail vehicle" or "LRV" for short) is arguably the most publically prominent feature of any LRT system. Everything about the remainder of the LRT system's infrastructure, facilities, and systems—including the track—is designed to make certain the LRVs can fulfill their function of transporting passengers in an efficient and expedient manner. However, LRVs come in a wide variety of designs, and it is essential to understand what the vehicle is before designing the track upon which it will run.

2.1.1 State-of-the-Art for Light Rail Vehicles

Major advancements have been made in LRV design since publication of the first edition of the *Track Design Handbook for Light Rail Transit*. These include but are not limited to the following:

- The near total adoption of low-floor and partial low-floor LRVs for virtually all new start projects and also for modernization of other existing light rail systems. Because of this, nearly all new vehicles have one or more trucks that have independently rotating wheels (IRWs) instead of conventional solid axles, adding significantly to the challenges in track design.
- Incorporation of crash energy management (CEM) principles in the design of vehicle carbodies. This has the benefit of not only increasing safety in collisions but also significantly reducing both overall vehicle weight and the loads applied by the wheels to the rails. This also reduces power consumption; a study for New Jersey Transit (NJT) concluded that a weight reduction per car of one metric tonne [about 1.1 short tons] can save approximately 24 million kWh of energy over a 30-year life cycle for a fleet of 100 cars, each operating 40,000 miles per year.[1], [2]
- Improved propulsion system, reducing weight, increasing performance and reliability, and reducing maintenance costs.
- Improved AC traction motor/parallel gear units of compact design that are resiliently mounted on the truck frame.
- New designs of resilient wheels that are both easier to install and reduce the unsprung mass to that of the steel tire, thus reducing high frequency shock and vibration of both truck and track components.[3]
- Adoption of LRVs with multiple (more than two) carbody sections by many transit agencies. Advantages include
    - Increased vehicle capacity
    - Reduced vehicle weight per passenger
    - Reduced number of main propulsion components
- Production of light rail vehicles very specifically intended for operation in public streets. These include not only "streetcars" that are somewhat smaller than the previous generation of LRVs but also incorporation of carbody design principles, such as enclosed front bumpers,

that make even larger LRVs more suitable for operation in areas with large volumes of pedestrians and motor vehicles.

- Articulated streetcar vehicles, with the trucks semi-rigidly attached to the carbody rather than swiveling relative to the carbody. Somewhat common overseas since the 1980s, these vehicles have now appeared in North America.
- Self-propelled Diesel Mechanical Unit (DMU) passenger railcars are now being operated in several North American cities. While these are not "light rail vehicles" as that term is defined in Chapter 1, they have many similar characteristics. Therefore much of this Handbook is applicable to systems using DMU vehicles.

Other changes in light rail vehicle design are occurring, and the list above could be obsolete in a very short time. For example, as of 2011, at least one manufacturer is actively marketing a streetcar-sized LRV for North American use that has "off-wire" operating capability. Such vehicles can operate for limited distances without an overhead catenary system by drawing power from an on-board energy storage unit (typically a battery). Off-wire capable vehicles seem very likely to become commonplace as the technology matures.

2.1.2 Vehicle/Trackway Interface

As vehicle technology continues to evolve, so does the complexity of the interface between the vehicles and the track. Even more than was the case when the first edition of this Handbook was published, there are few hard and fast rules about the relationships between vehicles and track on light rail transit systems. In spite of this lack of design consistency, there are several key vehicle-to-track and trackway parameters that the track designer must consider during design of light rail systems. These include

- Vehicle Weight (both empty and with full passenger load)
- Clearances
  - Required track-to-platform location tolerances to meet ADA requirements
  - Required clearance between cars on adjacent tracks considering car dynamics
  - Required route clearances (wayside, tunnel, bridge) considering car dynamics
- Wheel Dimensions
  - Wheel diameter, which can be very small in the case of low-floor vehicles and is virtually always smaller than that used on freight railroad equipment. Smaller wheel diameters produce higher contact stresses than larger wheel diameters, with resulting implications regarding rail corrugation and wear on both wheels and rail
  - Wheel profile or contour, including the wheel tread width, which must be compatible with the rail section(s) selected, particularly in the case of special trackwork
  - Wheel gauge, to ensure compatibility with the track gauge, including tolerances
  - Wheel back-to-back gauge that is compatible with flangeway dimensions and special trackwork check gauges
- Longitudinal Vehicle Forces on the Track
  - Maximum acceleration and associated tractive forces
  - Maximum/emergency deceleration from a combination of friction brakes, dynamic braking and electromagnetic track brakes, including the automatic application of sand

- Lateral Vehicle Forces on the Track
  - Maximum lateral forces resulting from all speed and curvature combinations
- Dynamic Vehicle Forces on the Track
  - Impact of car and truck natural frequencies
  - Impact of wheel flats or damaged wheels

It is essential that the track designer, the vehicle designer, and the designers of systems such as signals, catenary, etc., coordinate and cooperate to achieve compatibility between the LRT system components under all operating conditions. These interactions can be facilitated by generating a comprehensive design criteria manual for any new LRT system and keeping it updated with ”as-built” information as the project is developed, constructed, and operated.

It is generally inadvisable to design a new light rail line around the characteristics of only one make and model of light rail vehicle since doing so may limit choices for subsequent vehicle procurements as the system expands and matures. A transit system guideway may remain unchanged for a century or more, during which time it would not be unusual for three or more cycles of vehicle procurements to occur. Instead, it is recommended to consider a universe of candidate LRVs from several manufacturers and develop a fictitious “composite” LRV that incorporates the most restrictive characteristics of several cars, e.g., the longest, the widest, the one with the largest minimum radius capability, etc. In this fashion, the transit agency will not be forever restricted to using only one particular make and model of LRV. It also minimizes situations where parts of the track alignment are at the absolute minimum or maximum capabilities of the vehicle, a condition that is highly discouraged in any event.

When new vehicles are procured for an existing transit line, the vehicle must be specified to operate on the existing track unless a concurrent rehabilitation and upgrading of the old guideway is proposed. When an existing transit line is extended, the track standards for the extension must accommodate both the old rolling stock and any new vehicles that might be procured.

2.2 LIGHT RAIL VEHICLE DESIGN CHARACTERISTICS

2.2.1 Introduction

Light rail vehicles are built in a variety of designs and dimensions. In almost all cases, they are capable of being operated in coupled trains. Modern LRVs are generally much larger and heavier than their streetcar predecessors and can have axle loads just as large as, or even larger than, so-called "heavy rail" transit vehicles. Notably, the modern streetcars used in one U.S. city actually have slightly higher axle loadings than the light rail vehicles also used there.

Light rail vehicles vary in the following design characteristics:

- Unidirectional versus bi-directional
- Non-articulated versus articulated and, for the latter, the location(s) and configuration of the articulation joints
- 100% high-floor versus partial low-floor (typically 70% or less) versus 100% low-floor
- Overall size (width, length, and height)
- Truck and axle positions
- Weight and weight distribution

- Suspension characteristics
- Performance (acceleration, speed, and braking)
- Wheel diameter and wheel contour
- Wheel gauge

These characteristics must be considered in the design of both the vehicle and the track structure.

2.2.2 Vehicle Design

2.2.2.1 Unidirectional/Bi-Directional

Nearly all of the legacy streetcar systems in North America that survived up through the 1960s used unidirectional vehicles, most often the Presidents Conference Committee (PCC) streetcar. Such "single-end" cars had operator's controls in the forward end, doors on the right side, and a single trolley pole current collector at the rear. At the end of the line, cars negotiated a turning loop and ran to the opposite terminal. Because these vehicles could negotiate curves with centerline radii as small as 35 feet [10.7 meters], the amount of real estate needed for a turning loop was relatively small, usually only a single urban building lot. Transit companies typically found that the expense of buying properties and building loops was small compared to the savings associated with not having to maintain duplicate sets of control equipment in "double-end" trolley cars.

Current designs of high-capacity light rail vehicles have much larger minimum radius limitations and the amount of real estate that is required to construct a turning loop is much greater. Accordingly, while a few European light rail lines continue to use single-end, single-sided vehicles that require turning loops, most contemporary LRVs have control cabs in both ends and doors on both sides. These cars can advantageously reverse direction anywhere that a suitable crossover track or pocket track can be provided. This arrangement is usually more economical than the turning loop in terms of real estate required and has become the norm for most modern light rail transit systems. Crossovers and pocket track arrangements can often be sited within the confines of an ordinary double-track right-of-way and do not require the supplemental property acquisition needed for turning loops.

The following are some of the factors that should be considered when evaluating single-end versus double-end light rail vehicles:

- Systems with stub-end terminals at either one or both ends of the line or at any intermediate turnback location will require bi-directional vehicles.
- Bi-directional vehicles with two operating cabs and doors on both sides of the vehicle will cost more than a single-end LRV with only one cab and doors on only one side.
- For slow speed movements in a yard or under an emergency situation, many single-end LRVs have a "back-up controller" in the rear of the car, often hidden behind a panel or under a seat.
- Unless equipped with doors on both sides, single-end LRVs require that all station platforms be located on the same side of the tracks. Having doors on both sides of the vehicle provides the capability of having stations on either or both sides of the track, regardless of whether the vehicle has one operating cab or two.

- Single-end vehicles that have doors on both sides can be coupled back-to-back resulting in a double-end train.
- The choice of single-end versus double-end vehicles may have an impact on how yard and shop facilities are laid out. This in turn will affect the real estate requirements for that facility and hence its location. The yard location in turn may have a direct effect on the system operating plan.
- Double-end vehicles typically have more uniform wear of the wheels since the leading axle on each truck changes at the stub-end terminals. Single-end vehicles often develop thin wheel flanges on the leading axle of each truck while the flanges on the trailing axles incur relatively little gauge face wear. This directly affects the frequency and cost of wheel truing and ultimately wheel replacement.
- From a civil engineering perspective, stub-end terminals are less costly compared with the loops because, as noted above, of the land costs and other local space restrictions. Trackwork costs for a stub-end terminal versus a loop could be similar or greater depending on the configuration and amount of special trackwork associated with any terminal station, passing tracks, or storage tracks. Train control system costs are nearly certain to be greater for a stub-end terminal than for a loop terminal.
- Stub-end terminals have construction and maintenance costs associated with special trackwork and train control systems that differ from those of loop tracks. The designer must evaluate options based on life cycle costs.
- Dwell times for a loop terminal are appreciably less than those for a stub-end terminal, which can be advantageous at terminals with extremely close operating headways.
- If double-end cars are selected, it is still possible to have loops at some terminals should local conditions make that choice advantageous.
- Loop tracks are more likely to be sources of noise than stub-end terminals, possibly impacting both the wayside community and patrons alike. The crossover track movements associated with a stub-end terminal are more likely to be a source of ground-borne vibration, particularly if a double or “scissors” crossover is used.
- Loop tracks at an intermediate turnback point will require a crossing diamond, which is more likely to be a source of noise and vibration than the ordinary frogs in the crossover tracks associated with a center pocket track.
- If there is a reasonable probability that a line might be extended beyond some initial terminal location, a stub-end track arrangement—and hence double-ended vehicles—would usually be the logical choice.
- Stub-end tracks provide greater flexibility for vehicle storage during off-peak hours.

2.2.2.2 Non-Articulated/Articulated

The earliest electric streetcars in the 1880s were four-wheeled single truck vehicles. Streetcar ridership quickly outgrew the capacity limitations of such vehicles, and eight-wheeled double truck streetcars were common by 1900. Often, these larger cars would pull a trailer car for even more capacity. The first articulated streetcars appeared in the United States about the time of World War I, often by splicing together two older single truck cars, and later as three-truck vehicles

functionally very similar to high-floor, articulated LRVs of today. The objective of this evolution in vehicle design was to maximize not only passenger capacity but also the number of passengers carried per operating employee since labor costs, then as now, were a high percentage of the cost of transit operation.

That trend has continued up through the present with the result that multiple-section light rail vehicles have reached unprecedented lengths. Today, with the exception of legacy and heritage streetcar operations and three light rail systems that bought new rolling stock in the 1980s, all new and modernized North American light rail systems are using articulated cars with two, three, or more carbody sections. Two-section articulated LRVs, which were the most common design when the first edition of the *Track Design Handbook for Light Rail Transit* was published, are now being purchased only for those LRT lines that require a 100% high-floor car to match high-platform stations.

The development of LRVs with multiple-carbody sections (up to seven sections in the case of trams purchased in Budapest, Hungary, in 2007) was driven by the same issues as a century ago—carrying more passengers with fewer operating employees. Multiple-carbody vehicles also have fewer motorized trucks per passenger and thereby provide substantial energy savings. Several North American systems are following this trend. Toronto ordered new five-section streetcars in 2008. Dallas Area Rapid Transit, following a trend started in Europe, modified older two-section, high-floor light rail vehicles to add a low-floor center section. New Jersey Transit has investigated adding two additional sections to their current fleet of three-section, 70% low-floor cars.[4]

Where two body sections meet, a turntable and bellows arrangement connects the sections, allowing continuous through passage for passengers from one end of the car to the other. In the case of high-floor LRVs, a single such arrangement, centered over a truck of conventional design, is used to connect two carbody sections. Low-floor LRVs require two such articulations—one on each side of the center truck and center section of the carbody—since there is no room for the turntable above the special trucks required under low-floor cars. This usually results in a very short carbody section at each low-floor truck.

Particularly in the case of low-floor LRVs, there are many variations on articulation joints, as each LRV manufacturer has devised its own specific design. These hardware variations can affect vehicle clearances since the pivot points of the articulation can be a considerable distance off of the centerline of sharply curved track. Variations in center section design also affect steering and relative roll, which might have some affect on vehicle curving and rail wear, thus influencing rail steel selection, track gauge, and track superelevation. The track designer has little control over this, but the problem is more difficult with low-floor vehicles using independently rotating wheels than with conventional high-floor vehicles equipped with solid axles.

Existing systems contemplating a change to longer vehicles must consider overall train length and the impact that the revision might have on existing station platforms. Longer cars might require either a reduction of the number of vehicles in a train or lengthening existing platforms. One major LRT system in the United States initially designed their underground LRT stations for four-car trains of conventional two-section high-floor LRVs. When they added low-floor vehicles, trains had to be limited to three of the longer low-floor cars because the subway station platforms

could not be economically lengthened. Longer vehicles can affect other infrastructure and systems as well, particularly the layout of equipment within the light rail vehicle maintenance shop.

There is a common misconception that articulated light rail vehicles can negotiate sharper curves than a rigid body car. This is not true. Rigid cars can negotiate curves that are as sharp, and even sharper, than an articulated vehicle. However, rigid cars are limited in both length and passenger capacity, primarily because the lateral clearances required in curves increase dramatically as the distance between the trucks increases. Where lateral clearances are not an issue, rigid body cars can be appreciably cheaper to procure and maintain than articulated cars of similar passenger capacity; however, this is a distinct exception to the normal circumstances.

In North America, modern non-articulated light rail vehicles are used only in Philadelphia, Buffalo, and Toronto, but, as of 2010, those fleets, which are all high-floor designs, are in their third decade of operation. Outside of North America, the light rail system in Hong Kong and several cities in the former Soviet Bloc have continued to purchase rigid body cars, most likely for reasons peculiar to those systems. Therefore, while thousands of single unit, single-end trams, many of them of designs derived from the North American PCC car, still operate around the world, it is virtually certain that the LRVs for any new system will always be high-capacity, multiple-section, articulated cars.

2.2.2.3 High-Floor/Low-Floor LRVs

2.2.2.3.1 Introduction

Getting passengers safely and expeditiously onto and off of light rail vehicles at stations has always been an issue. Time spent at stations—"dwell time"—can be a significant percentage of the overall running time from terminal to terminal. For a conventional "high-floor" light rail vehicle, with steps at the doors that are internal to the vehicle, the delays inherent in climbing up and down steps adds significantly to the dwell time. The various measures necessary to comply with the Americans with Disabilities Act Accessibility Guidelines (ADAAG) means even more delay before such LRVs can resume forward motion.

Level boarding from the platform to the vehicle is clearly the best way to accommodate the mobility-challenged transit rider. Level boarding also reduces station dwell times by making it easier and quicker for all riders, mobility-challenged or not, to board and alight from the LRV. Because of these advantages, heavy rail metro systems have always used level or near level boarding from high level platforms. Following that example, several light rail systems built during the 1980s, in both North America and Europe, incorporated level boarding from high level platforms, largely eliminating the need for steps.

The problem with high level platforms is that they usually can fit alongside of the tracks only if the light rail line is in an exclusive guideway such as a subway tunnel, an aerial structure, or a private right-of-way. High platforms that are the full length of the train (usually no less than 200 feet/60 meters for a two-car train) are generally impractical where the LRT guideway is in an urban street. Urban locations often also have insufficient space for vertical circulation elements to get passengers from street and sidewalk level up to a station platform that would usually be 3 feet

[0.9 meter] higher. Moreover, a two- or three-car long high platform will often be very intrusive on the urban streetscape, as well as quite expensive.

Because of such issues, light rail systems that were constructed in the 1980s and early 1990s and included extensive operations in city streets typically used high-floor LRVs that were equipped with steps for patrons to board from sidewalk level. A variety of methods were used to get mobility-challenged persons on and off the vehicles, with mini-high platforms being the usual choice. However, these arrangements were generally less than fully satisfactory. Some means of providing level boarding for all riders without resorting to full-length high level platforms was desired.

2.2.2.3.2 Low-Floor Cars—General

In response to these issues, low-floor light rail vehicles were developed. In a low-floor car, either the middle portion or all the vehicle floor is positioned a short distance above top of rail. A typical dimension is 300 to 350 mm [about 11.7 ¾ to 13 ¾ inches]. This enables station platforms to be little more than sidewalks that are just slightly higher than normal above the street surface, making them much more practical for construction in congested urban areas.

Since about 1995, the partial low-floor car (often called a "70% low-floor" LRV) has become the preferred design for North American light rail transit systems that need level boarding from low platforms. The partial low-floor car has some middle portion of the LRV at the lower elevation while the ends of the car are at normal high-floor car elevation. The doors are usually all in the low-floor section of the car and the high-floor areas at the ends of the car are accessed by interior steps. The low-floor area usually represents approximately 70% of the total length of the car, hence the common name. (Boston's Type 8 LRVs are a notable exception; clearance limitations in the Green Line tunnels substantially restricted the truck center distance so that the low-floor portion of each car is only about 60% of the overall length.)

One advantage of a 100% low-floor LRV is that the low profile of the cab and windshield increases the probability of eye contact between the operator and persons on the trackside. A corresponding advantage to a high-floor or 70% low-floor LRV is that the operator's higher seating provides a better view of the trackway ahead, which could be an advantage in some traffic situations.

One possible issue with low-floor cars is that they maintain very close clearance to rails. With worn-out wheels, the vertical clearance between the underside of truck-mounted equipment and the plane of the top of rail can be a little as 35 mm [1 $^{3}/_{8}$ inches]. This could affect the use of some trackwork and signal system appliances mounted between the rails. The vehicle clearance also must be considered in design of tracks for hilly terrain, where the radius of the vertical curve over the crown of the street must be large. On one project, the low underclearance of the vehicle limited the height of discontinuous floating slabs that could be used, where maximum mass is needed for vibration control.

2.2.2.3.3 Low-Floor Car Truck Design

The ends of the 70% low-floor car, including the operator's cabins, are generally at the same height as a high-floor car, allowing trucks of conventional design under the ends of the car. But it is not possible to use conventional trucks beneath the low-floor portions of the car because the

floor would be lower than the elevation of solid axles. The usual resolution is to use trucks that do not have conventional solid axles extending from wheel to wheel. Instead, the four wheels are each connected directly to a u-shaped frame that passes beneath the floor. Each wheel, lacking a mechanical connection to another, therefore rotates independently and is naturally called an independently rotating wheel (IRW). As an alternative to IRW trucks, at least one manufacturer has developed a truck using conventional solid axles connecting very small diameter wheels. This design also ramps the floor of the articulation body section slightly above that of the floor by the doors. However, small diameter wheels will have a smaller contact patch with the top of rail and thereby increase wheel/rail contact stresses, possibly increasing rail wear and corrugation rates.

Because of constrained space, these special truck designs beneath the center sections of 70% low-floor LRVs are generally non-powered. Propulsion is provided only at the conventional trucks under the ends of the car. However, 100% low-floor cars must provide propulsion at trucks under the low-floor, and carbuilders have come up with several ingenious, albeit complex, methods for doing this. Because of this complexity, 70% low-floor cars using conventional power trucks have generally been considered more reliable than 100% low-floor cars. Nevertheless, the 100% low-floor LRV has been almost exclusively adopted for new vehicle purchases by in-street tramway type operations in Europe and also by some of the stadtbahn-type operations. As of 2010, the first 100% low-floor LRV specified in North America was being produced for Toronto Transit Commission. The Toronto cars are also specified to negotiate a 36-foot [11-meter] radius curve. The degree to which the carbuilder succeeds in meeting the Toronto requirements may radically change preferences for light rail vehicle design.

As of 2010, the lowest 100% low-floor LRV was the Vienna Ultra-Low-Floor (ULF) car, with the floor a mere 200 mm [about 8 inches] above the top of the rail. The traction motors of the ULF car are mounted vertically within the articulation sections. As of 2010, this design has not been adopted elsewhere.

The conventional trucks that are under the end body segments of 70% low-floor cars rotate with respect to the carbody. By contrast, the trucks under 100% low-floor LRVs generally do not rotate and are, for all practical considerations, rigidly fixed to the carbody. This configuration has resulted in vehicle designs that are radical departures from high-floor and partial low-floor designs and vehicles that have significantly different steering and curve negotiation characteristics.

2.2.2.4 Carbody Strength, Crashworthiness, and Mass

2.2.2.4.1 Introduction

Up until about 1970, there were no codes or standards for the overall strength requirements of a transit vehicle carbody that were fully based in engineering principles. Beginning about that time, the usual requirement in specifications became that the carbody needed to accept, without structural failure, a longitudinal static “buff load” equal to two times its own mass. This was known as the “2-g standard,” although it was never actually codified as a mandatory requirement except in the State of California.[5] Under the 2-g standard, if the vehicle weighed 125,000 pounds [556 kilonewtons] it needed to have a minimum buff strength of 250,000 pounds [1,112 kilonewtons]. Naturally, the addition of more steel to make the carbody stronger also increased

its mass, with the result that new transit cars were much heavier than their predecessors. This extra weight had impacts on power consumption, structure design, and track design.

2.2.2.4.2 Crash Energy Management

In response to those issues and following the lead of European LRV manufacturers, crash energy management (CEM) principles began to be incorporated into the design of light rail vehicles for North American use. CEM, which has been used in the automotive industry for decades, recognizes that designing the vehicle body to collapse in a controlled and predictable manner during a collision is better at minimizing injuries to the vehicle occupants than just merely making the carbody stronger.

Beginning with a procurement of light rail vehicles for New Jersey Transit in the mid-1990s, CEM design principles began to replace the old 2-g criterion.[1], [2], [3] Subsequently, new standards were developed on both sides of the Atlantic. In Europe, European Norm (EN) 15227—Railway applications—Crashworthiness requirements for railway vehicle bodies,[6] was implemented in 2008. A companion standard is EN 12663—Railway applications—Structural requirements of railway vehicle bodies.[7]

In North America, the American Society of Mechanical Engineers developed ASME RT-1—Safety Standard for Structural Requirements for Light Rail Vehicles.[8] ASME RT-1, which is somewhat more restrictive and conservative than EN 15227, became effective in 2010. An updated edition is expected to be issued by ASME in 2014. As of 2010, for North American applications, either the ASME RT-1 or EN 15227 are voluntary (as was the old 2-g criterion) unless they are adopted and codified by either federal or state regulation.

The European Norm and ASME RT-1 differ in several respects, and the latter is generally more rigid. For example, ASME RT-1 includes a collision scenario at 25 mph [40 km/h] while the equivalent EN 15227 test is performed at 25 km/h [16 mph]. Hence, vehicles designed to just meet the European Norms will likely not comply with ASME RT-1.

The 100% low-floor cars for Toronto Transit Commission's legacy streetcar system were specified to meet EN 15227, with a slightly higher Category 4 speed, since ASME RT-1 existed only in draft form at the time of the procurement in 2008. The cars for Toronto's Transit City program (underway as of 2010) were similarly specified under EN 15227 rather than changing from one voluntary standard to another.

Since nearly all North American LRVs are designed and at least partially built overseas, the lack of consistency between European and North American standards increases procurement costs. The resultant heavier vehicles also have long-term ramifications concerning operating energy costs and loading and wear and tear on the track structure. As of this writing, it is unclear whether consistency between the North American and European standards will be possible. What does seem clear is that many of the lightweight LRVs that are common in other parts of the world are unlikely to be used in the United States, particularly on any project that utilizes federal funding.

However, this situation is evolving. As of early 2011, revisions to ASME RT-1 that would eliminate all structural requirements that are inconsistent with European standards and may

unnecessarily increase the procurement costs are under consideration. Whether those changes will be adopted in whole or part cannot be predicted, and rail transit design practitioners must therefore keep current with evolving best practices.

2.2.2.4.3 LRV Bumpers

A key feature of many modern LRVs is a front end bumper that is designed around crash energy management principles. The bumper typically extends from a few inches above the rails to the floor level of the LRV. The bumper is designed to rotate upward, revealing the LRV coupler. The coupler itself, which traditionally extended out an appreciable distance beyond the front of the LRV, is now hinged and can be folded back behind the closed bumper. The bumper conceals the traditional anticlimber as well as the coupler, but is not primarily intended to be merely cosmetic. Because of the CEM design, in the event of a collision, the bumper actually minimizes damage to any motor vehicles. It also makes it far less likely that an automobile would become wedged beneath the front of an LRV. Similarly, the bumper makes it more likely that a struck pedestrian will be pushed aside instead of being pulled beneath the front of the LRV. As of 2011, bumpers are not universal on new light rail vehicles, but it seems likely that they will become a common feature for any LRVs that have extensive operations in public streets.

2.2.2.4.4 Vehicle Mass

As an example of what CEM principles can mean to carbody mass, it is useful to compare the 70% low-floor LRVs built for New Jersey Transit with those delivered to Santa Clara County (San Jose), California. The latter were constructed to the 2-g criterion under California PUC regulation 143-B while the former were designed around CEM principles. The same carbuilder produced both cars, and they have the same overall dimensions, performance, and capacity. The California car has a maximum wheel load at AW2 loading that is 540 pounds [245 kg] greater than that of the New Jersey LRV, a difference of 3.2 tons [2.9 metric tonnes] per car. The difference will result in appreciable propulsion energy cost savings over the life cycle of the New Jersey Transit car as well as less loading and wear and tear on the track.

Table 2.2.1 compares the vehicle mass per unit of floor area between comparable 100% low-floor and 70% low-floor cars from selected European and North American cities. The difference averages about 100 kg/m^2 [about 20.5 lb/ft^2]. For an LRV that is 27.5 meters [90 feet] long and 2700 mm [8.9 feet] wide, this amounts to 7425 kg (16,390 lb) of additional weight that the vehicle must carry around through its entire service life, with implications for both energy consumption and loads applied to the track structure. In addition, the 100% low-floor vehicle may produce lower wheel/rail contact stresses than those produced by the 70% low-floor vehicle.

One part of the difference in vehicle mass between low-floor and conventional articulated vehicles with solid axles is due to the deletion of the traditional truck. However, a major part of the difference is the different standards under which the cars were specified. Of the two North American 70% low-floor cars in Table 2.1, only the New Jersey Transit car was designed around CEM principles. Several of the European vehicles predate EN 15227 and EN 12663 and their degree of compliance with those standards is unclear. It is also very likely that most of these vehicles may not comply with ASME RT-1; therefore, for purposes of potential North American application, they may be irrelevant. Some might also argue that some of these vehicles are "trams" as opposed to "light rail vehicles." As noted in Chapter 1, European light rail operations typically don't make such distinctions between vehicle types.

Table 2.2.2 shows some of the characteristics of modern light rail vehicles operating in North American cities as of 2010. The table is not intended to be a comprehensive reference of every vehicle or every system now operating but rather an illustration of the rather wide array of vehicles that a track designer might encounter on any given project. Because light rail systems are constantly purchasing new cars and retiring older cars (and, in some cases, selling retired cars to other systems), the table is merely a snapshot of a dynamic condition. Track engineers working on designs for any transit system, including those listed below, should obtain up-to-date information on the agency's current LRV fleets before commencing any design.

Table 2.2.1 Relative mass of 100% vs. 70% low-floor LRVs

100% Low-floor LRVs		70% Low-floor LRVs	
City	Weight – lbs/ft^2 [Mass – kg/m^2]	City	Weight – lbs/ft^2 [Mass – kg/m^2]
Lille	98 [480]	Kassel	93 [456]
Socimi	64 [312]	Valencia	107 [521]
Strasbourg	90 [440]	NJ Transit	114 [558]
Munich (Munchen)	99 [482]	Rostock	95 [462]
Chemnitz	71 [345]	Vienna (Wien) "T"	100 [489]
Frankfurt	106 [516]	Portland	132 [644]
Turin (Torino)	96 [470]	Grenoble	133 [650]
Vienna (Wien) ULF	80 [388]	Bochum	100 [486]
		Leipzig	107 [523]
		Heidelberg	97 [473]
AVERAGE	88 [429]	**AVERAGE**	108 [526]

Table 2.2.2 Light rail vehicle characteristics matrix (2010 data)

	CITY Carbuilder/Model	DELIVERY YEAR	WEIGHT AW0 lbs	MAXIMUM WHEEL LOAD lbs	LENGTH Feet	CARBODY CONFIGURATION	FLOOR LEVEL
1	Baltimore						
	ABB	1989/1995	108,000	12,000	95	6-axle 2-carbody	High
2	Boston						
	KS	1982	85,000	9,350	74	6-axle 2-carbody	High
	Breda	2000	86,300	9,500	74		50% Low
3	Buffalo						
	Tokyu	1985	71,000	11,000	66'-10"	4-axle 1-carbody	High
4	Calgary						
	Siemens SD 160	1999/2008	89,600	9,800	81'5"	6-axle 2 carbody	High
5	Charlotte						
	Siemens S 70	2004/2008	96,800	10,700	93'6"	6 axle 3 carbody	70% low
6	Cleveland						
	Breda	1982	91,300	9,800	80'	6-axle 2-carbody	High
7	Dallas						
	KS 1	1998	108,000	11,600	92'6"	6-axle 2-carbody	High
	KS 2	2007	140,000	15,176	123'6"	8-axle 3-carbody	Low
8	Denver						
	Siemens SD 100	1995	88,000	9,650	81'6"	6-axle 2-carbody	High
	Siemens SD 160	2008				6-axle 2-carbody	
9	Edmonton						
	Duewag U 2	1982	67,300	7,900	79'8"	6 axle 2-carbody	High
	Siemens SD-160	2009	91,700	9,960	81'4"	6-axle 2-carbody	High
10	Houston						
	Siemens S 70	2004	98,500	10,950	96'6"	6-axle 3-carbody	70% low
11	Los Angeles						
	Nippon	1992	98,000	10,700	89'	6-axle 2-carbody	High
	Siemens SD100	1993				6-axle 2-carbody	High
	Siemens P2000	1999	98,000	10,700	89'	6-axle 2-carbody	High
	Breda 2550	2008	89,000	9,970	90'	6-axle 2-carbody	High
12	Minneapolis						
	BBD Flexity	2004	99,180	10,940	94'	6 axle 3-carbody	70% low
13	New Jersey						
	Kinki Sharyo	2000	93,500	10,350	90'	6 axle 3-carbody	70% low
	BBD (DMU)	2005	119,000	18,000	102'	6 axle 3-carbody	70% low
14	Norfolk						
	Siemens S 70	2008	96,800	10,720	93'6"	6-axle 3-carbody	70% low
15	Philadelphia					4-axle 1-carbody	
	City	1982	57,300	6,200	50'	Single end	High
	Suburban	1982	59,500			Double end	High
16	Phoenix						
	Kinki Sharyo	2008	102,000	11,100	91'5"	6-axle 3-carbody	70% low
17	Pittsburgh						
	Duewag /CAF	1984/R2005	97,000	10,500	84'8"	6-axle 2-carbody	High
	CAF	2004	100,000	10,740	84'8"	6-axle 2-carbody	High
18	Portland						
	Bombardier	1986	92,150	10,200	89'1"	6-axle 2-carbody	High
	Siemens SD 660	2000	109,000	11,700	92'0"	6-axle 2-carbody	High
	Siemens S 70	2009	99,000	10,990	96'6"	6-axle 3-carbody	70 % low
	Skoda Inekon	2006	56,000	9,813	66'0"	4-axle 3-carbody	50% low
19	Sacramento						
	Siemens SD 100	1991	77,175	8,690	79'6"	6-axle 2-carbody	High
	CAF	2003	93,735	10,190	83'9"	6-axle 2-carbody	High
	UTDC	1989	98,700	10,740	88'6"	6-axle 2-carbody	High
20	St. Louis						
	Siemens SD100-1	1993	90,390	10,080	89'5"	6-axle 2-carbody	High
	Siemens SD100-2	2001	93,000	10,290	89'5"	6-axle 2-carbody	High
21	Salt Lake						
	Siemens SD 100	2002	88,000	9,650	81'5"	6-axle 2-carbody	High
	UTDC	1989	98,700	10,740	88'6"	6-axle 2-carbody	High
	Siemens S70	2010	TDB	TDB	TDB	6-axle 3-carbody	60% Low

Table 2.2.2 Light rail vehicle characteristics matrix (2010 data) (continued)

	CITY Carbuilder/Model	DELIVERY YEAR	WEIGHT AW0 lbs	MAXIMUM WHEEL LOAD lbs	LENGTH Feet	CARBODY CONFIGURATION	FLOOR LEVEL
22	Seattle Kinki Sharyo	2008	102,000	11,200	95'0"	6-axle 3-carbody	70% low
23	San Diego Siemens U2 Siemens SD100 Siemens S 70	 1989 1996 2005	 71,800 88,000 95,500	 8,250 9,650 10,540	 79'8" 81'5" 90'7"	 6-axle 2-carbody 6-axle 2-carbody 6-axle 3-carbody	 High High 70% low
24	San Francisco Breda	1998	78,000	8,630	75'0"	6-axle 2-carbody	High
25	San Jose Kinki Sharyo	2001	99,980	10,890	90'0"	6-axle 3-carbody	70% low
26	Toronto UTDC CLRV UTDC ALRV	 1982 1987	 51,000 78,600	 8,612 8,750	 52'6" 77'6"	 4-axle 1-carbody 6-axle 2-carbody	 High High

No attempt was made to include vintage or heritage streetcars in Table 2.2.2 since they come in so many versions. Further, since even the newest of the vintage PCC streetcars still operating in the United States will be 60 years old in 2012, it is an open question how long the use of any such vintage equipment in daily revenue service can be sustained. Modern low-floor streetcars, which can directly comply with ADAAG without resorting to wheelchair lifts and/or ramps and which could also easily be constructed with a faux antique appearance, would seem to be a more rational choice for new streetcar programs. As is the case with any modern light rail car, the track designer should inquire as to the characteristics of any vintage streetcars that might be proposed to occasionally operate over the system so they can be accommodated in the design of both track alignment and trackwork.

2.3 VEHICLE CLEARANCES

This article discusses the dimensional characteristics of the light rail vehicle. This includes not only the static vehicle at rest, but also the additional dynamic movements the LRV can make due to both resiliency and possible failures in the vehicle suspension system. The result is a definition of the vehicle dynamic envelope (VDE). The VDE, plus additional factors, defines the track clearance envelope (TCE), which sets the minimum distances between the centerline of track and any infrastructure alongside of the track as well as the minimum distances between tracks. Because the TCE includes elements that are unrelated to the vehicle, it will be discussed in detail in Chapter 3.

2.3.1 Vehicle Clearance Envelopes

Clearance standards for various types of railroad cars are well documented by the use of graphics or "plates." For railroad equipment, one standard is the common "Plate C." Any car whose static dimensions fit within the limits established on Plate C can travel virtually anywhere on the North American railroad system. Transit systems do not have similar national standards. Therefore, transit vehicle manufacturers must develop vehicles that fit within the clearance requirements of the system for which the car is intended. Conversely, transit system designers should, whenever possible, configure the infrastructure so as to allow clear passage of as broad a universe of candidate LRVs as possible. While manufacturers can, in theory,

build cars to any dimension, it is usually more economical to choose vehicles that are already in production or have at least been engineered. Therefore, the facility designer of a new system should establish a composite vehicle clearance envelope that accommodates vehicles from several manufacturers to maximize competitive bidding and then design the system to accommodate those clearances.

The composite vehicle clearance envelope considers both the static and dynamic outlines of the vehicles under consideration. The static outline is the cross-sectional shape of the car at rest on tangent level track. The dynamic outline includes the allowable movement in the suspension system due to vehicle pitch, roll, yaw, and curving characteristic. The manufacturer develops the actual dynamic outline for their transit vehicle so as to fit within the owner's clearance restrictions.

In addition, as the vehicle passes through curved track, the lateral excursions of the carbody will vary depending on the static plan shape of the vehicle, the distance between the trucks, and the amount of curvature. To establish clearances along the right-of-way, a vehicle dynamic clearance envelope must also be developed. Using the vehicle dynamic outline along with the associated track components, track tolerances, wear limits of the components, and a running clearance zone, the track clearance envelope can be established.

LRV procurement specifications may include the following requirements related to clearances:

- A dynamic envelope as established in the project's Manual of Design criteria.
- Minimum clearance under any car component under worst wheel and suspension condition.
- The minimum track curve radius.
- The maximum allowed curve offset and minimum carbody shift in the tightest track curve radius under worst track conditions and/or with maximum superelevation.
- Demonstration that the horizontal clearance (gap) and vertical match to station platforms is in compliance with ADAAG. The latter require that passengers step down from the car floor onto the station platform when alighting from the vehicle even with the worst situation of wear on both wheels and rail.
- Gap between vehicle door sill and platform edge, which may affect wheelchair access.

Trackform design may influence the clearance envelope; ballasted track may shift with time while direct fixation and embedded track will not.

For additional information on vehicle clearances, particularly the track and wayside issues that affect the structure gauge and the swept path of the LRV through curves, refer to Chapter 3, Article 3.4.

2.3.2 Vehicle Static Outline

The static outline of an LRV is based on plan and cross-sectional views showing its dimensions at rest, including many elements as discussed below.

2.3.2.1 Vehicle Length

When considering the length of a light rail vehicle, it is important to distinguish between the actual length of the carbody and its length over the coupler faces as follows:

- Over Coupler Face—The coupler is the connection between LRVs that operate together. It extends beyond the front of the car structure. The length over the couplers becomes a consideration for determining the requisite length of facilities such as station platforms and storage tracks for coupled and uncoupled trains.
- Over Anticlimber or Bumper—The anticlimber is a ribbed bumper at floor elevation positioned at the structural end of the car. In the event of a collision between two LRVs, the anticlimbers on each car will interlock and, as the name implies, thereby reduce the possibility of one LRV climbing over the floor level of the other during a collision. The length of the vehicle over the anticlimbers was traditionally used to determine clearances, but the current generation of light rail vehicles often conceals the anticlimber behind a movable bumper. Regardless of whether the LRV is equipped with a bumper or a visible anticlimber, the positions of the outer corners of the device with respect to the track centerline and the vehicle trucks will often define the swept path of the vehicle toward the outside of any curve.

When considering the length of a light rail vehicle, it is important to distinguish between the actual length of the carbody and its length over the coupler faces.

Another important longitudinal dimension, one that generally does not affect clearances but can be a significant design factor, is the distance from the leading edge of the first door on the LRV to the rear edge of the last door on the car (or the last door on a multiple-car train). Occasionally, while doing track alignment at a station, providing a segment of tangent track that is the full length of a train may not be possible. However, if only the door-to-door dimension is used to define the ADAAG-compliant platform edge, it may make the difference between being able to provide a station at a key location versus having no station at all. This topic is discussed further in Chapter 3.

2.3.2.2 Distance between Truck Centers

The distance between adjacent truck pivot points determines the overhang of a car's midsection for given track curvature. This "truck center" distance is a key factor in determining the extent of the vehicle's swept path toward the inside of the curve. A vehicle with a long truck center distance will have a greater "mid-ordinate" clearance excursion than one with a shorter truck center distance. Conversely, a vehicle that has a truck center distance that is relatively short will usually have a large "end-overhang" clearance to the outside of the curve.

In the case of the center truck of a low-floor LRV, the pivot points are not coincident with the center of the truck. As a result, they will be located some distance to the outside of the centerline of the track as the car passes through a curve, affecting both the mid-ordinate and end-overhang distances. (Notably, during curving, longitudinal and transverse forces may induce rotation of the center truck/carbody section, increasing angle of attack, gauge face wear, and noise and may affect ballasted track alignment stability.)

2.3.2.3 Distance between End Truck and Anticlimber or Bumper

This dimension and the carbody end taper (if any) determine the overhang of the front of the car toward the outside of the curve for a given track curvature.

2.3.2.4 Carbody Width

The width of the LRV carbody is determined by several factors:

- In the case of any LRV that will be operating in mixed traffic in a street, it generally should comply with the legal maximum widths for motor vehicles. There can be some latitude on this since, unlike a large rubber-tired vehicle such as a truck, the path of the LRV is absolutely predictable. See Chapter 12 for additional discussion on this point.
- The transit agency requirements regarding the total number of passengers seated versus standing, the number and arrangement of seats, specified human factors for the width of the single seats and double-seats, and allowances for wheelchairs of standard size.
- Total vehicle wall thickness.
- In the case of an existing LRT system procuring new vehicles, any existing clearance restrictions may limit several vehicle dimensions, including width. Vehicle procurement specifications for existing systems replacing legacy rolling stock typically need vehicles no wider than 8.33 to 8.83 feet [2540 to 2690 mm] in width so as to match existing clearances.[9]

In some cases, the sides of the carbody are tapered, rather than vertical, so that the car is narrower at the ceiling than it is at floor level. This taper partially compensates for vehicle roll and keeps the dynamic clearance envelope smaller. The widest point on some rail cars is actually located at window sill level so as to maximize shoulder room for seated passengers.

A few North American systems can accommodate wider than normal light rail vehicles. The Breda LRVs in San Francisco are 9.0 feet [2745 mm] wide. Cleveland's Breda LRVs are 9.3 feet [2835 mm] wide while Baltimore's ABB light rail vehicles, which were designed to operate on tracks shared with freight trains, are 9.6 feet [2925 mm] wide.[10] Trams as narrow as 2400 mm [7.9 feet] are operated on some European systems where close clearances cannot allow wider cars. Such narrow cars are not recommended for new operations since their passenger capacity is significantly less than standard width vehicles.

2.3.2.5 Carbody End Taper

The plan view configuration of the end of a light rail vehicle is usually not square. Instead, it is tapered, usually over the length of the operator's cabin. The principal reason for this is neither aesthetics nor aerodynamics but rather to reduce the dynamic excursions of the ends of the LRV as it passes through curved track. **Figure 2.3.1** illustrates a typical three-section articulated LRV passing through a tight radius curve. Note how the amount of taper at the ends of the car reduces the clearance requirements to the outside of the curve. If the carbody maintained the same width all the way to the end of the car, the vehicle excursions to the outside of the curve would be much greater. Some vehicles have even more taper so that clearances to the outside of the curve are actually controlled by the carbody width at the rear of the operator's cab and not at the nose of the car. The reduced width of the front of the cab still provides sufficient room for the operator's dashboard and other equipment.

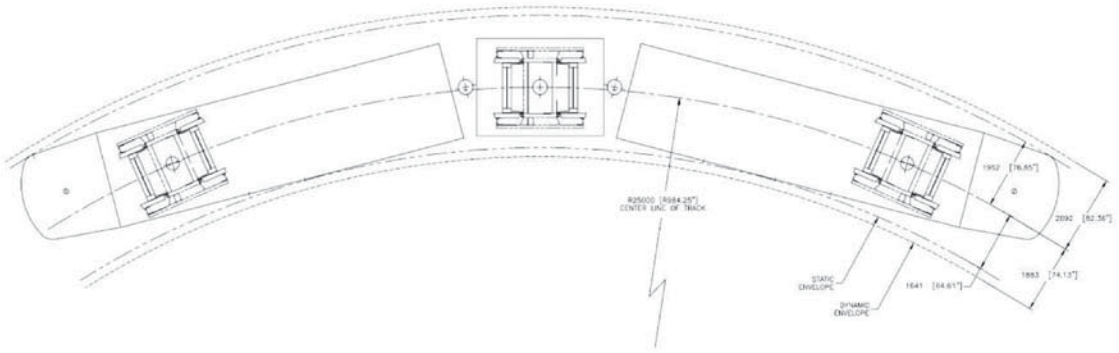

Figure 2.3.1 Three-section 70% low-floor LRV in an 82-foot [25-meter] radius curve

The ideal situation clearance in curves on a double-track route is to design the end taper and select the truck centers and pivot point locations so as to make the mid-ordinate and end-overhang clearances at equal distances from the track centerline. This permits placement of the catenary poles exactly halfway between the two tracks. The designers of the vehicles for one rail transit project were able to balance these so that on an 82 foot [25 meter] radius curve, the end-overhang and the mid-ordinate differed by only about ¼ inch [6 mm]. This new LRV also fits within the clearances of the PCC streetcars that formerly operated on a portion of that reconstructed and expanded light rail system.

2.3.2.6 Other Static Clearance Factors

On most light rail vehicles, the overall width is governed by the external rear view mirrors, which are mounted on the corners of the car outside of the motorman's cabin. Notably, the mirrors are only a clearance control at the elevation where they are mounted. Trackside objects that are higher or lower than the mirrors can sometimes be placed closer to the track.

Some LRVs are now equipped with rear facing cameras, which permit the operator to monitor multiple locations along the length of the vehicle or train from a display screen on the dash. There usually will be several cameras on each side of the car with some facing forward as well as backwards. Some jurisdictions prohibit video displays that can be seen by a motor vehicle operator, and waivers of those regulations may be required.

The cameras are much smaller than the mirrors they replace and each might extend out beyond the face of the vehicle only half the distance required for a mirror, thereby making the clearance outline of the vehicle appreciably narrower. The cameras are also mounted somewhat to the rear of the motorman's cabin and so do not widen the vehicle body at the ends of the car. This can significantly reduce the "end-overhang" vehicle clearance requirements to the outside of curved track, making it possible to take full advantage of the LRV body end taper.

The doors on some light rail vehicles have thresholds which project out some distance beyond the sides of the carbody. These are sometimes designed to be "sacrificial" should they collide with a platform edge. Projecting thresholds are sometimes seen on systems that have a mixed vehicle fleet where the actual width of one or more series of rail cars are narrower than others. This permits both wide and narrow vehicles to service the same platforms.

The geometric center of the plan view of a rail vehicle truck in curved track will not be coincident with the centerline of the track, but rather shifted some distance toward the inside of the curve. The magnitude of this shift will vary depending on the axle spacing of the truck, the radius of the curve, the lateral position of the truck relative to the rails, and any skew the truck may have assumed relative to the track. For LRV trucks with axle spacings less than about 6 feet 6 inches [2.0 meters] the shift is negligible for curves with radii greater than 300 feet [91 meters]. It can be a factor for sharper curves and/or longer axle spacings.

2.3.3 Vehicle Dynamic Envelope/Outline

The dynamic outline of the car is more significant to the track alignment designer than the static outline. The vehicle dynamic envelope (VDE) of an LRV describes the maximum space that the vehicle may occupy as it moves along the track. The dynamic outline or "clearance envelope" includes many factors due to the normal actions of the vehicle's suspension system, such as carbody roll (side sway) and lateral movement between stops. The dynamic outline also includes lateral freeplay between wheels and rail with both in their maximum wear condition as well as abnormal conditions that may result from failure of suspension elements (e.g., deflation of an air spring).

The development of the VDE is typically the responsibility of the vehicle designer and begins with the cross-sectional outline of the static vehicle. The dynamic outline of the vehicle is then developed by making allowances for carbody movements that occur when the vehicle is operating on level tangent track. These movements represent the extremes of carbody displacement that can occur for any combination of rotational, lateral, and vertical carbody movements when the vehicle is operating on level tangent track.

The following items are typically included in the development of the VDE:

- Static vehicle outline
- Dynamic motion (roll) of springs and suspension/bolsters of vehicle trucks
- Vehicle suspension side play and component wear
- Vehicle wheel flange and radial tread wear
- Maximum truck yaw (fishtailing)
- Maximum passenger loading
- Suspension system failure
- Wheel and track nominal gauge difference
- Wheel back-to-back mounting and maintenance tolerance

In addition, some projects include allowances for the following:

- Rail fastener loosening and gauge widening during revenue service
- Dynamic rail rotation

However, since these two factors are not under the control of the vehicle supplier and could also vary considerably with trackform, it is recommended that these factors not be included in the VDE

but instead be addressed by the track designer as part of the track construction and maintenance tolerances. If the vehicle designer does include track factors in the VDE, that fact needs to be clearly documented. Whoever adds the track tolerances must utilize relatively liberal maintenance tolerances and not the typically stringent construction tolerances in the determination of the VDE.

Typical values for vehicle-based maintenance factors include the following:

- Nominal wheel gauge-to-track gauge freeplay: 0.405 inch [10.5 mm]
- Lateral wheel flange wear: 0.3 inch [7.5 mm]
- Vertical radial wheel wear: 1 inch [25 mm]

The VDE is usually represented as a series of exterior coordinate points with the reference origin at the track centerline at the top-of-rail elevation.

The static vehicle outline is generally not used in track design except for the establishment of station platforms and associated station trackwork design at these locations.

The dynamic outline is compiled for tangent track with zero cross-slope in the rails. Track curvature, superelevation, and maintenance tolerances are considered separately and will be discussed in Chapter 3 at Article 3.3.4.

Any project will actually have two dynamic envelopes to consider:

- The first will be a proposed or provisional dynamic envelope that is developed as a part of the LRV procurement specification. This will be based on the characteristics of the hypothetical composite LRV. The procurement specification will typically include language such as: *"*The vehicle shall be designed to operate within the dynamic envelope under all condition of wear or failure other than structural failures.*"*

- The second envelope will be the actual dynamic envelope for the vehicle purchased. It will be provided by the selected vehicle manufacturer and indicate its conformance to the specification (or, in some cases, situations where a waiver of some portion of the provisional envelope is requested).

In **Figure 2.3.2**, the outer lines indicate the dynamic envelope stipulated in one procurement contract while the inner dotted lines show the supplier's compliance with the specified limits.

The vehicle dynamic outline is merely a two-dimensional cross section of the car illustrating its extreme movement due to factors related to the car itself. As the vehicle and its dynamic envelope pass along the track, they generate a three dimensional shape known as the "swept path." The characteristics of the swept path will be discussed in Chapter 3 at Article 3.8.1.3.1.

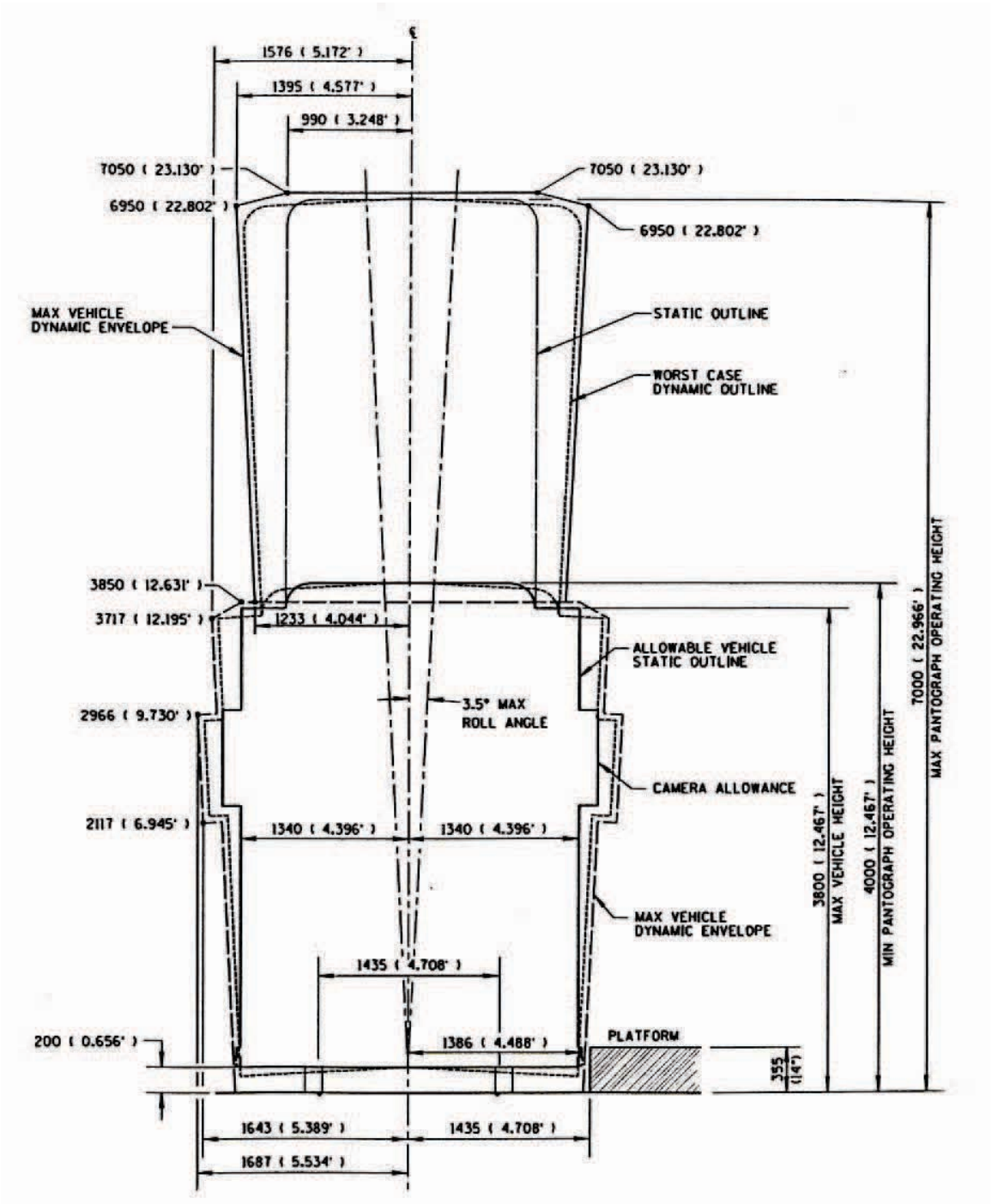

Figure 2.3.2 Typical LRV dynamic envelope

2.3.3.1 Vehicle Components Related to Vehicle Dynamic Envelope

The vehicle dynamic envelope is influenced by both the as-fabricated characteristics of the vehicle, particularly its suspension system, and possible wear and/or failure of vehicle subassemblies. These factors include

- Primary/secondary suspension systems
- Maximum roll/lean/sway
- Maximum lean due to total failure of all truck components
- Wheel tread and flange wear

Air springs (also known as air bags) are a common element in the secondary suspension system. They serve multiple functions, including keeping the floor both reasonably level and matched to the station platform height regardless of the number of passengers on board. The air springs on each truck are interconnected by lines which include balancing valves. The balancing valves detect changes in pressure in one air bag versus the other and automatically make adjustments. In the case of a sudden loss of pressure in one bag, the balancing valve will automatically deflate the other. This prevents a sudden change in the LRV's center of gravity that might otherwise result from one side of the carbody abruptly rising to the mechanical limits—an event that could unload one or more wheels and lead to a derailment or cant the vehicle excessively and conflict with tunnel wall appurtenances.

2.3.3.2 Track Components Related to Vehicle Dynamic Envelope

Various issues related to the track will affect the magnitude of the dynamic excursions of the LRV. These include the following:

- Track superelevation/crosslevel
- Wheel gauge-to-track gauge lateral clearance/freeplay
- Construction tolerances and maintenance tolerances for track surface, crosslevel, and alignment
- Maintenance tolerances for rail head wear and gauge face wear

Typically, the only factor in the list above that is included in the vehicle dynamic envelope would be the design value of freeplay between the track gauge and the wheel gauge. The other factors are not under the control of the vehicle supplier and therefore should instead be addressed by the track designer. Sometimes the vehicle supplier will include track-related factors in its calculated VDE, but those numbers can include unrealistically stringent assumptions as to the track maintenance tolerances that can be achieved. So as to avoid double-counting such issues, the track designer should back out any track-related tolerances that may be in the vehicle supplier's VDE and substitute values that are consistent with the transit agency's maintenance track maintenance standards.

2.3.3.3 Vehicle Clearance to Wayside Obstructions and Other Tracks

It is not unusual to have clearance restrictions on an LRT line that cannot be either simply or economically altered. In such cases, the track designer should coordinate with the vehicle and structural designers to ensure that the vehicle dynamic envelope considers these limitations so that adequate clearances result. Vehicle dynamics are governed by the car's suspension system(s) and, therefore, indirectly by numerous factors of track and vehicle interaction. For multiple-track situations, multiple clearance envelopes must be considered. Overlapping of the

vehicle dynamic envelopes from adjacent tracks obviously must be avoided. The resulting requirements will dictate minimum track centers and running clearances for tangent and curved track, including construction and maintenance tolerances as input to the track alignment calculations.

In general, the absolute minimum tangent track centers for vehicles of normal width (e.g., 2650 mm / 8.7 feet) for rigid trackforms (direct fixation or embedded) are 13 feet 6 inches [about 4.15 m] with a catenary pole between the tracks. If the poles are outboard of the tracks, 11 feet [about 3.35 m] is the typical minimum spacing. Tangent track center spacing for ballasted track is typically 6 inches [15 cm] greater than those for rigid trackform track due to greater allowances for construction tolerances and shifting of the tracks over time. Track curvature and superelevation increase these dimensions. These issues are discussed further in Chapter 3, Article 3.8.

2.3.3.4 Platform Clearances

One clearance requirement that can be difficult for vehicle manufacturers is keeping the dynamic envelope at platform height from intersecting the edge of the platform. Since ADAAG requires the horizontal gap between the static vehicle and the platform to be 3.0 inches [76 mm] or less, the fully dynamic vehicle might actually strike the platform. In the case of high-floor LRVs adjacent to a high level platform, interference between the platform edge and the vehicle dynamic envelope is virtually inevitable. This is largely because the vehicle roll center is typically about 2 feet [approximately 0.6 meter] below the platform surface.

However, LRVs virtually never actually strike a high platform edge because it is extremely unlikely that the vehicle and track factors that might lead to full excursions to the limits of the dynamic envelope will ever occur simultaneously. The use of a rigid trackform (e.g., either embedded or direct fixation track) and/or scrupulous maintenance of ballasted track surface and crosslevel and horizontal alignment can minimize the track contribution to vehicle dynamics. On the vehicle side, thresholds that project beyond the face of the vehicle and are designed to be “sacrificial” can minimize damage to both the vehicle and the platform edge.

Low-floor LRVs have very little chance of striking a low platform edge because the platform surface is typically a few inches [centimeters] below the carbody roll center as shown in Figure 2.3.2. Hence, while the platform clearance might still be reduced by carbody lateral translation, roll will not increase the encroachment.

See Article 2.9 in this chapter and Chapter 3, Article 3.8.3 for additional discussion concerning the interface between LRVs and station platforms.

2.3.3.5 Pantograph Height Positions

When discussing the height of a light rail vehicle, two conditions must be considered:

- Roof—The roof of an LRV is typically curved, with the highest dimension at the car centerline. However, the LRV pantograph, when deployed, obviously establishes the maximum car height. In the case of high-floor LRVs, the pantograph is the highest point on the car even when in the “lock-down” position. Low-floor LRVs, which have much more equipment on the roof (since there is little room under the floor), sometimes have some equipment sitting higher than the pantograph. However, the overall height of the

car with the pantograph locked down is typically only of concern in the design of maintenance shop infrastructure, such as the entrance door to a paint booth, where the LRV would usually be pushed or towed by other equipment. Lock-down clearances would only be a consideration along revenue service track if the LRV has "off-wire" operating capability.

- Pantograph Operation—Light rail facility designers are typically interested in the absolute minimum clearance between the top of the rail and an overhead obstruction, such as a highway bridge. This dimension must accommodate not only the pantograph when operating at some working height above lock-down, but also the depth of the overhead contact wire system. The minimum pantograph working height above lock-down includes an allowance for pantograph "bounce" so that lock-down does not occur accidentally.

Maximum pantograph height is typically the concern of only the vehicle and overhead catenary system (OCS) designers, unless the light rail guideway must also accommodate railroad freight traffic and attendant overhead clearances. If railroad equipment must be accommodated, the clearance envelope will be dictated by AREMA-recommended practices, state regulations, and the standards of the freight railroad involved. The minimum height of the trolley wire above a freight track will be much higher than the minimum height above an LRT-only track. See Articles 3.8.4 and 11.5.3 for additional discussion of this topic.

2.4 VEHICLE-TRACK GEOMETRY

The most demanding light rail transit alignments are those running through established urban areas. Horizontal curves must be designed to suit existing conditions, which can result in curves below a 25-meter (82-foot) radius. Vertical curves are required to conform to the existing roadway pavement profiles, which may result in exceptionally sharp crest and sag conditions.

LRVs are specifically designed to accommodate severe geometry by utilizing flexible trucks, couplings, and mid-vehicle articulation. Articulation joints, truck maximum pivot positions, coupler-to-truck alignments, vehicle lengths, wheel set (axle) spacing, truck spacing, and suspension elements all contribute to vehicle flexibility. The requirements for the truck to accommodate, within reasonable limits, free movement in three planes are defined in the vehicle procurement specification. Guidelines for these factors are included in the APTA *Manual of Standards and Recommended Practices for Rail Passenger Equipment,* RP-M-009-98 Recommended Practice for New Truck Design. [12] The torque the truck exerts against free turning is critical for resistance against derailment. Light rail carbody/truck connections that use either a ball bearing slewing ring or a king pin, without side pads, generally have good horizontal free movements. Air spring suspensions generally provide satisfactory free roll and yaw movements. Truck-related submittals from the vehicle supplier may include proof of compliance with the Truck Swivel Index (TSI), a factor calculated in accordance with Koffman's Formula, a guideline developed by British Rail in the 1970s.

The track designer must take into account the vehicle characteristics defined in the articles below when developing alignments. The values associated with these characteristics are developed and furnished by the vehicle manufacturers. The manufacturer of vehicles supplied to existing systems must meet the existing minimum geometrical requirements of the system.

2.4.1 Horizontal Curvature—Minimum Turning Radius of Vehicle

The minimum turning radius is the smallest horizontal radius that the LRV can negotiate. In some cases, the value may be different for a single LRV versus two or more coupled into a train or for a fully loaded LRV versus an empty one. However, the inclusion of curves in a track layout that can only be negotiated by a single vehicle is absolutely not recommended since operating personnel may not remember the restriction, particularly during an emergency situation such as when an inoperable LRV must be pushed off the revenue line by its follower. The vehicle procurement specifications will therefore typically stipulate only the minimum radius that multiple-car trains of LRVs must be able to negotiate. The LRV supplier will typically be required to provide submittals that demonstrate that the proposed vehicle can negotiate the tightest curve under full design load without any binding in the trucks, articulation joints, or couplers. A specification for one LRV procurement stipulated:

> *The coupler and draft gear shall allow under emergency conditions, a three vehicle train with an AW3 passenger load, operating at degraded dynamic performances, to push or tow an inoperable similar train consist loaded to AW3 without damage to the coupler, over all grades and curves of [the system].*[9]

Often, the minimum operable multi-vehicle train length requirement will be much longer than the consists actually required for revenue service. This is so as to accommodate shop and yard movements and other exigencies. Such long consists will occasionally have some impact on track alignment. One vehicle specification stipulated:

> *The vehicle shall be capable of multiple unit operation in consists up to six vehicles. A normal operation is up to three vehicles.*[9]

2.4.2 Vertical Curvature—Minimum Sag and Crest Curves

The minimum vertical curvature is the smallest vertical curve radius that the LRV can negotiate. The maximum sag and crest values are typically different, with the sag value being more restrictive. Vehicle builders describe vertical curvature in terms of either the radius of curvature or as the maximum angle in degrees through which the articulation joint can bend. The trackway designer must relate those values to the parabolic vertical curves typically used in alignment design.

When new vehicles are procured for an existing system, they must be able to negotiate the most restrictive current track condition. Conversely, when existing vehicles will be used on a new extension of an existing system, the new track must accommodate the existing vehicle's capabilities. The vehicle procurement specification will include requirements related to specific track conditions, be they existing or proposed.

2.4.3 Combination Conditions of Horizontal and Vertical Curvature

The car builder may or may not have a graph that displays this limitation. If a route design results in significant levels of both parameters occurring simultaneously, the design should be reviewed

with potential LRV suppliers to establish mutually agreeable limits. The following is a typical example from one vehicle specification:

> *Reverse vertical curve: A two-vehicle consist shall be capable of negotiating a reverse vertical curve section involving: first, a crest of 250 m [820 feet] and a sag of 350 m [1150 feet], separated by a tangent section of 13 m [43 feet]; and second, a crest and sag curve of 500 m [1640 feet] separated by no tangent track.*[9]
>
> *Compound curves: A two-vehicle consist shall be capable of negotiating a compound [horizontal and vertical] curve involving: first, a 25 m [82 feet] radius horizontal curve and a 500 m [1640 feet] radius vertical curve, either crest or sag; second, a 27 m [89 feet] radius horizontal curve and a 350 m [1150 feet] radius sag curve; and third, a 29 m [95 feet] radius horizontal curve and a 250 m [820 feet] radius crest curve.*[9]

Alternatively, a set of plan and profile drawings can be included as an appendix in the vehicle procurement documents giving complete geometric information, including gradients, civil design speeds, and track superelevation.

2.4.4 Vertical Alignment—Maximum Grades

The maximum grade that a light rail vehicle can ascend is limited by the electrical and mechanical limits of the propulsion system. The maximum grade that an LRV can descend is limited by the braking system. Both climbing and descending are constrained by the limits of adhesion between the wheels and the rails. Tractive effort between wheels and rails is dependent on the amount of vehicle weight on powered axles and, generally speaking, light rail vehicles that have all axles powered can more reliably climb steep grades than cars with some number of non-powered axles. Braking is virtually always available on all wheels, powered or not. However, descending steep grades can sometimes be a greater issue than climbing the same hill since a high percentage of the braking effort is required to slow the vertical descent and hence not available to retard horizontal movement.

Generally, grades of unlimited length up to about 6% to 7% are not a problem for any light rail vehicle. Above that the operational impacts should be reviewed, including:

- The tractive and braking characteristics of the LRV in normal operation.
- Situations where a disabled LRV (or train of LRVs) is being pushed or towed by another train. The critical situation might not be pushing the disabled vehicle train up the grade, but rather controlling the descent when going down the hill.
- The possibility of any lubricants on the rail running surface, particularly grease that might have migrated from some nearby curve and unintentionally lubricated the rail running surface.

Grades of up to 10% are possible, and some legacy streetcar lines, using cars with all axles powered, were even steeper. However, wheel-to-rail slippage can occur on any gradient during inclement weather conditions, such as when snow, ice, or wet and/or oily leaves are on the rail.

Slippage may result in rail burns during both acceleration and braking and wheel flats during braking.

Light rail vehicles have always been equipped with sanders, activated by the operator to drop dry sand on the rail and thereby increase friction between wheel and rail. Modern vehicles with slip/slide detection will also automatically dispense sand when required. Sand will therefore accumulate along steeply graded tracks and also in station areas. The sand will mix and bond with other contaminants on the trackway (including rail lubricants and friction modifiers) and wash downgrade to the lowest points on the track structure. Ideally, the track design should provide for this contamination to wash away harmlessly before it can become a path for stray currents and corrosion, but a comprehensive housekeeping program to keep accumulated sand from becoming a problem is generally necessary.

Combinations of steep gradients, small radius horizontal curves, and sharp vertical curves are found on many light rail lines. One LRT line in the eastern United States has an 82-foot [25 meter] horizontal curve on a down grade of 6% followed by a sag vertical curve with a radius of about 1640 feet [500 meters]. At the other end of that vertical curve is a short up grade of 7% leading to a crest vertical curve followed by the junction turnout to another route. Legacy streetcar lines often had alignments that were even more convoluted. While such tortuous track alignments are possible, they tax the capabilities of the vehicle, slow down transit operation, require much higher than normal maintenance, are usually sources of high noise and vibration, and cause poor ride quality. They therefore are generally not recommended unless absolutely nothing better is possible within the project budget. The track alignment designer should work closely with all other project disciplines, including the vehicle engineers, so as to be certain that any complicated track alignments do not create any intractable problems for other members of the design team.

2.4.5 Maximum Allowable Track Twist

Truck equalization refers to the changes in individual wheel loading that occur when one wheel on a two-axle truck moves above or below the plane of the other three wheels. If a wheel is unloaded significantly, it may climb the rail and derail. The truck needs to be sufficiently limber so as to maintain roughly equal vertical load on all four wheels regardless of any such twist and avoid unloading.

Several situations can result in twist that can unload one wheel of a truck:

- Misalignments in the track surface such as a low rail joint that has dropped some measurable distance below the plane of the rails.
- Track superelevation transitions where the profile of one rail is rising relative to the other.
- Deliberate twist in tangent or curved track such as an embedded track section where normally crowned pavement (required for drainage) transitions to either a level or superelevated section.

LRV truck equalization must be compatible with the maximum expected track vertical surface misalignment to prevent conditions that can cause a derailment. The following is a typical

specification for the maximum wheel unloading when one wheel is leaving the horizontal plane—such as when being lifted by the outer rail on spiral curve with superelevation:

> *Lifting or lowering any wheel on a truck 38 mm (1.5 inches) shall not cause the load to change on any wheel of that truck by more than 50% with the vehicle on level tangent track and under an AW0 load. Loss of contact shall not result between any of the wheels and the rail when raising or lowering one wheel on a truck up to 50 mm (2 inches).*[9]

The dimensions above provide a considerable factor of safety so as to avoid routinely loading the truck to its mechanical limits and are unlikely to occur in track. For example, an LRV truck with axle centers of 6 feet [about 1.8 meters] that is negotiating a spiral with a superelevation raise rate of 0.20% (about ¾ inch in 31 feet or 19 mm in 9.45 meters), will have the leading outside wheel raised by only 0.15 inches [4 mm]. Even if the track surface had substantially deteriorated, it is unlikely that track twist over the length of a truck would ever be more than ¾ inch [19 mm]. However, the equalization parameters above are for a static test. A vehicle operating at track speed will not be as limber; therefore, track twist must be restricted.

The allowable twist is usually expressed either as a percentage as noted above or as a ratio *y:x*. with *y* being an amount of superelevation and *x* being the length over which it is achieved, using the same units for both. A common limit is 1:400 as in 1 inch of superelevation in 400 inches/33.33 feet [roughly 25 mm in 10 meters]. However, some low-floor vehicle manufacturers have requested 1:500 as a track twist design limit. One U.S. transit agency that was having problems with center truck derailments on their partial low-floor LRV has established a maintenance standard of approximately 1:425.

These ratios sharply contrast with the capabilities of legacy rolling stock with more limber truck designs. The PCC car, which was deliberately designed to operate on abysmal track, can deal with track twist of about 1:150. More to the point, the new twist limit figures are more restrictive than one of the formulas that has traditionally been used for determining minimum spiral lengths for LRT. That topic is discussed at length in Chapter 3, Article 3.2.5; however, the point to be made here is that track designers should obtain specific information from their peers on the vehicle side of the project regarding acceptable values of track twist. Ideally, the vehicle designers should provide three figures:

- A desirable twist ratio for track design.
- A minimum twist ratio for track maintenance. (This would be somewhat less restrictive and indicate the point at which corrective track surfacing should be undertaken.)
- An absolute minimum twist ratio to be used as a safety limit. This value, which may be speed dependent, would indicate that possible derailment is imminent unless corrective actions (either resurfacing of the track, speed reductions, or both) are taken.

“Jump frogs” as described in Chapter 6, Article 6.6.6, are becoming a popular item for seldom-used diverging movements at special trackwork and were once very common on legacy streetcar lines. These will raise one wheel of the truck a dimension equal to the height of the wheel flange, typically 1 inch [25 mm]. Operation over the diverging side of such frogs must be done at very slow speed so that the vehicle suspension system has time to respond to the truck equalization

requirements. If jump frogs are proposed on an LRT project, that fact should be clearly identified in the vehicle procurement documents.

2.4.6 Light Rail Vehicle Ride Quality

Light rail vehicle ride quality is defined in typical North American specifications as the capability to operate, at any speed up to the vehicle's maximum operating speed (MOS) and at any passenger loading, free from vibration and shocks, to the specified levels.

2.4.6.1 Vehicle Natural Frequency as a Factor in Ride Comfort

All of the light rail vehicle's equipment is required to be free from resonance. To achieve this, resonances must be damped, and the natural resonance frequencies of the carbody must be sufficiently removed from the secondary suspension resonance frequency. Most vehicle specifications include language such as the following:

> *The carbody natural frequency shall be 2.5 times the secondary suspension natural frequency.*

Vehicle specifications usually require that a dynamic and ride quality model should be developed using programs such as NUCARS or VAMPIRE and performance be proven via model predictions. The ride quality is evaluated according to ISO 2631, Mechanical vibration and shock—Evaluation of human exposure to whole-body vibration—Part 1: General requirements, Figures 2a-Vertical and 3a Horizontal.[13] In this case, the appropriate limit is the 8-hour fatigue limit to which the transit vehicle operator might be exposed. Transit patrons can be exposed to higher limits, as their exposure time would be considerably shorter. Note that the vehicle operator could be exposed to higher levels of vibration at the nose of the car than the patron would be at the center of the car.

The ride quality is tested with a vehicle in good operating condition, with new wheels on tangent track that has been maintained to a class appropriate for the test speed, at vehicle crush loading of AW3. For this condition, the accelerations experienced by the passenger should generally not exceed 0.315 m/sec^2 [about 1.0 ft/sec^2], which is equal to 0.03 g. Another test, with air suspension deflated, is performed to confirm safe train operations under a partial failure condition and should not exceed 0.620 m/sec^2 [about 2.0 ft/sec^2] or 0.06 g.

ISO 2631 does not specify specific test procedures. In the case of DMUs procured for one project, the tests were performed according to a European standard: UIC 518, Test and Acceptance of Railway Vehicles from the Point of View of Dynamic Behavior, Safety Against Derailment, Track Fatigue, and Quality of Ride.[14] This standard determines vehicle compliance considering track alignment design, track geometry, and related operating conditions such as the cant deficiency and speed.

2.4.6.2 Track Geometrics as a Factor in Ride Comfort

See Chapter 3, Article 3.2.4 for an extensive discussion concerning ride comfort as a factor in determination of characteristics of curved track, including speed, radius, superelevation, and spiral length.

2.5 VEHICLE STRUCTURAL LOADS

2.5.1 Static Vertical Loads

ASME RT-1[8] defines light rail vehicle weights as follows:

- AW0: Empty load: the weight of the vehicle ready to run with all mounted components, including full operating reserves of lubricants, windshield fluid, etc., but without crew and passenger load.
- AW1: Fully seated load: AW0 plus the crew and fully seated passenger load.
- AW2: System load: AW1 plus 4 passengers per meter^2 [3.3 per yd^2] in standing areas.
- AW3: Crush load: AW1 plus 6 passengers per meter^2 [5.0 per yd^2] in standing areas.
- AW4: Structural load: AW1 plus 8 passengers per meter^2 [6.7 per yd^2] in standing areas.

The mass of each passenger and crew member is stipulated as being 70 kg [154 lb], a figure that seems low at first glance, but makes allowances for children as well as adults of various statures. The AW4 loading is an extraordinary condition used only for the design of undertrack structures.

2.5.2 Wheel Loading Tolerance (Car Level)

While most light rail vehicles appear to be completely symmetrical at first glance, the arrangement of various parts of the underfloor and rooftop equipment means that the actual loads applied to each truck will vary. A typical vehicle specification includes the tolerances related to overall weight distribution between the three or more trucks and the maximum acceptable wheel load variation per truck basis.[2] While the numbers will vary, the following text is typical of the language found in vehicle procurement specifications for a three-truck articulated vehicle:

- *The vehicle weight supported at center truck shall be within the range of 25 to 30% of the total vehicle weight*
- *The difference in vehicle weight between the A end and the B end trucks shall not exceed 450 kg (1000 lb)*
- *The lateral imbalance (wheel to wheel at the same axle, and expressed as a moment rotating vertically about the center of the axle) shall not exceed 100 kg-m (8500 in-lb)*

2.5.3 Wheel Loading at Maximum Stationary Superelevation

Worst-case wheel/rail force is expected when a fully loaded (AW3) car stops on a maximum superelevated track structure. Car tilt will also add to the lateral and vertical forces on the lower rail. The vehicle's center of gravity projection when stationary on the maximum superelevation must be within the gauge of the tracks with a sufficient margin of safety. Typical practice is to keep it within the middle third of the track gauge; see Chapter 3, Article 3.2.4.1.

2.5.4 Unsprung Mass

Unsprung weight in the LRV trucks is a significant contributing factor to dynamic track loading and ground-borne vibration as these items are not isolated from the track by the vehicle's primary and

secondary suspension systems. The use of resilient wheels theoretically reduces unsprung mass to only the weight of the tire; however, the elastomeric elements of resilient wheels still need to be fairly stiff so as to keep the tire both circular and concentric with the axle. Hence, until relatively recent times, the axle and the gearbox were effectively unsprung mass.

Modern truck designs achieve further isolation of the traction motor and gearbox unit by resiliently installing them on the truck frame and having the axle floating in the gearbox's hollow output shaft, relying on a flexible coupling ("dog bones") to transmit torque to the wheel set.[3] The resilient wheel reduces truck shock and vibration, which is generally beneficial, but does introduce a resonance of the wheel set within the tire with a frequency of about 50 to 100Hz. The interaction among track stiffness, tire, wheel set, and truck frame is quite complicated and may vary considerably with design. This can be important with respect to track vibration isolation design.

2.5.5 Truck Design

Light rail vehicle truck design has evolved appreciably since the light rail renaissance of the 1990s. The trucks on those early vehicles incorporated many features that had been successfully employed on heavy rail metro vehicles—such as monomotor design (i.e., both axles powered by a single motor, rather than one motor per axle)—that proved to be ill-suited for light rail vehicles operating on very sharp radius curves. Current designs build on that experience and provide much better performance (including a significant margin of safety against derailment) due to the following features:

- Shorter wheelbase (spacing between axles), which generally facilitates curving but can increase the angle of attack in a curve. (All other things being equal, a longer wheelbase truck will require wider flangeways and wider track gauge than a truck with a short wheelbase.)
- Longitudinally resilient axle mountings/primary suspension with resilient metal inserts.
- Resilient axle mounts in the transverse direction to reduce the impact upon entering the curve.
- Reduced unsprung masses—resilient wheels and drive units.
- Very low turning resistance due to being connected to the carbody with a ball bearing slewing ring and king pin without side plates.

2.5.5.1 Motorized Trucks

Since the late 1990s, conventional power trucks have almost exclusively used AC traction motors and parallel helical gear units. These replaced the DC monomotors and hypoid gears commonly used on light rail vehicles up through the early 1990s. **Figure 2.5.1** illustrates a typical power truck such as might be used under either a high-floor LRV or a 70% low-floor LRV. Features shown include AC motors and parallel gear units that are fully suspended resiliently on the truck frame, resilient wheels, chevron primary and air spring secondary suspensions, center king pin connection to carbody underframe, disk brake installed on the gear exit shaft, track brakes, train-to-wayside and cab signaling antennas, and on-board wheel flange lubrication.

The power trucks beneath 100% low-floor cars are much more sophisticated since they require room for the low-floor passenger cabin to pass between the wheels and truck frame. **Figure 2.5.2** illustrates an outside frame truck design for narrow gauge track with the motors mounted longitudinally outboard of and between the wheels. The design powers both wheels on each side of the truck from a single motor, appreciably changing the way the truck interacts with the track compared with a conventional solid axle power truck.

Figure 2.5.3 illustrates a low-floor power truck with conventional solid axles. This design utilizes small diameter wheels—600 mm [23.6 inches], roughly 100 to 110 mm [about 4 to 4.5 inches] smaller than the wheels used on most LRV trucks. The carbuilder also places the floor in the articulation module higher than the floor in the main body sections, with a ramp between the areas.

Figures 2.5.2 through 2.5.3 are only a few of the many designs of low-floor power trucks that are on the market as of 2010. Some other designs utilize even more radical features such as individual "hub-mounted" motors on each wheel. The state of the art is advancing rapidly and truck designs such as those illustrated here may well become obsolete. The reader is encouraged to review current trade publications and literature available on manufacturers' websites for up-to-date information specific to the vehicles under consideration for a project.

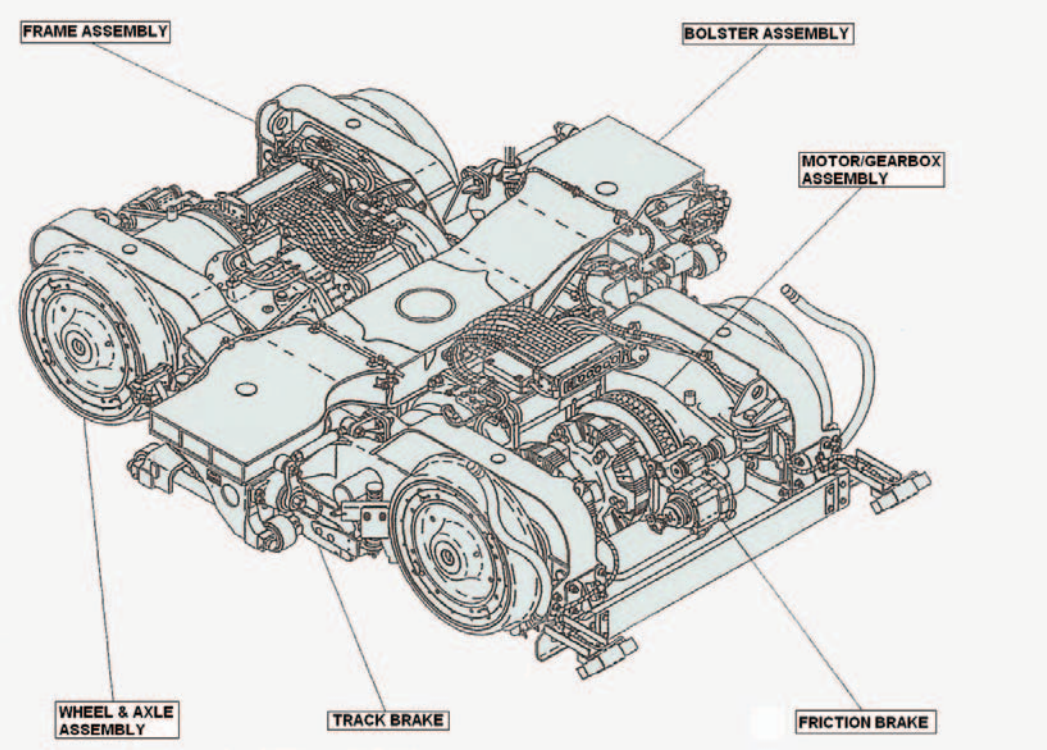

Figure 2.5.1 Kinki Sharyo power truck for 70% LRV

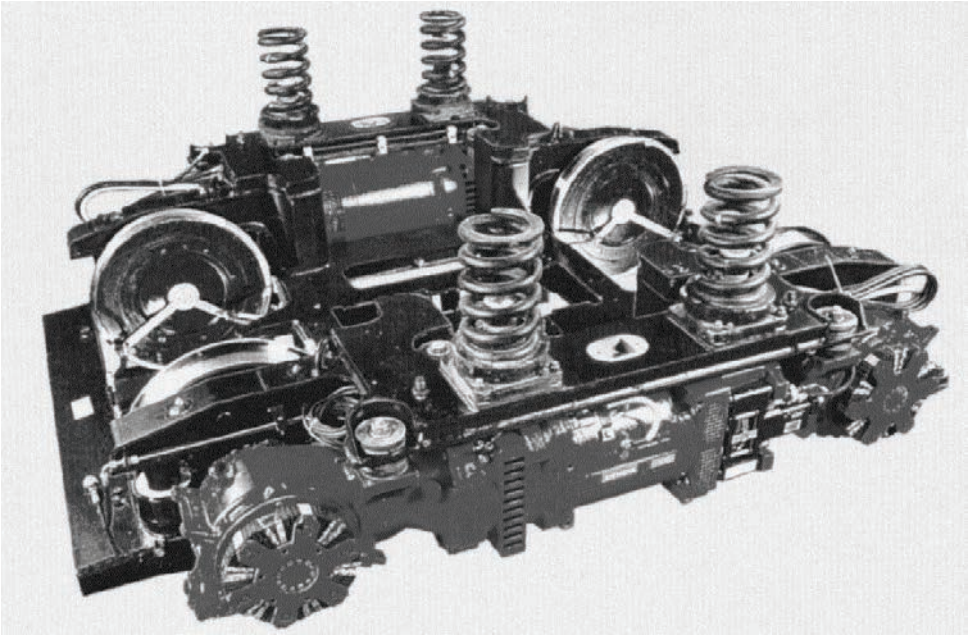

Figure 2.5.2 Siemens power truck for a Combino 100% low-floor narrow gauge LRV

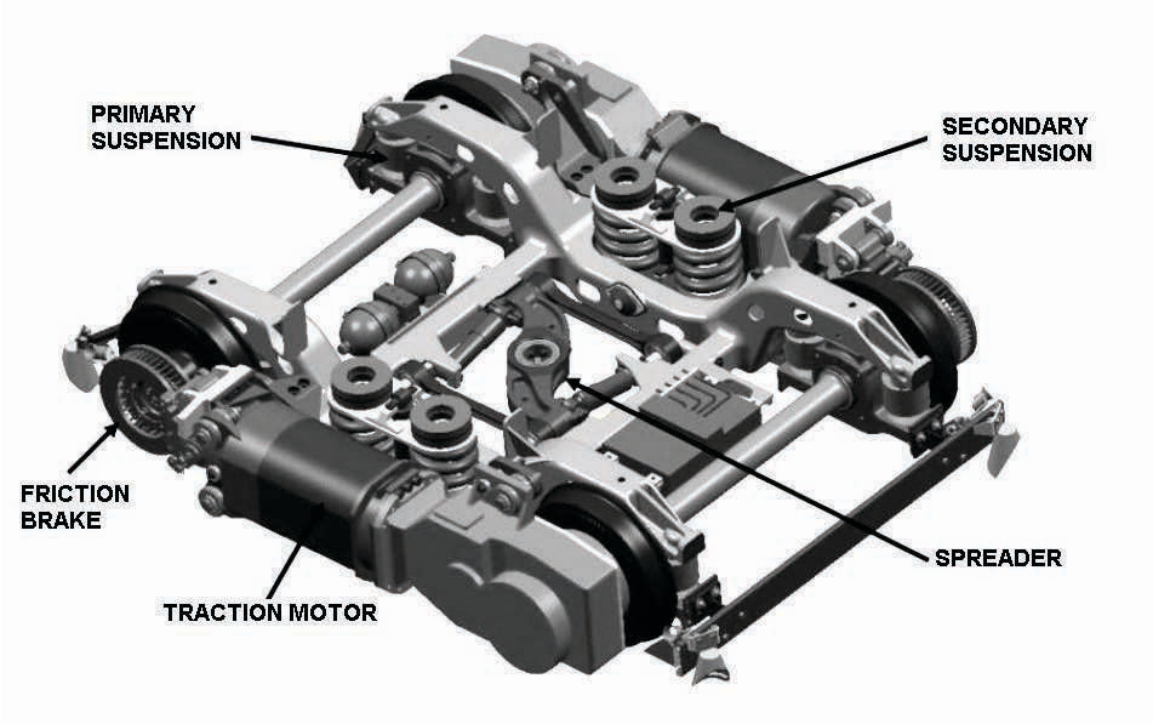

Figure 2.5.3 Bombardier Flexity Outlook power truck for 100% low-floor LRV

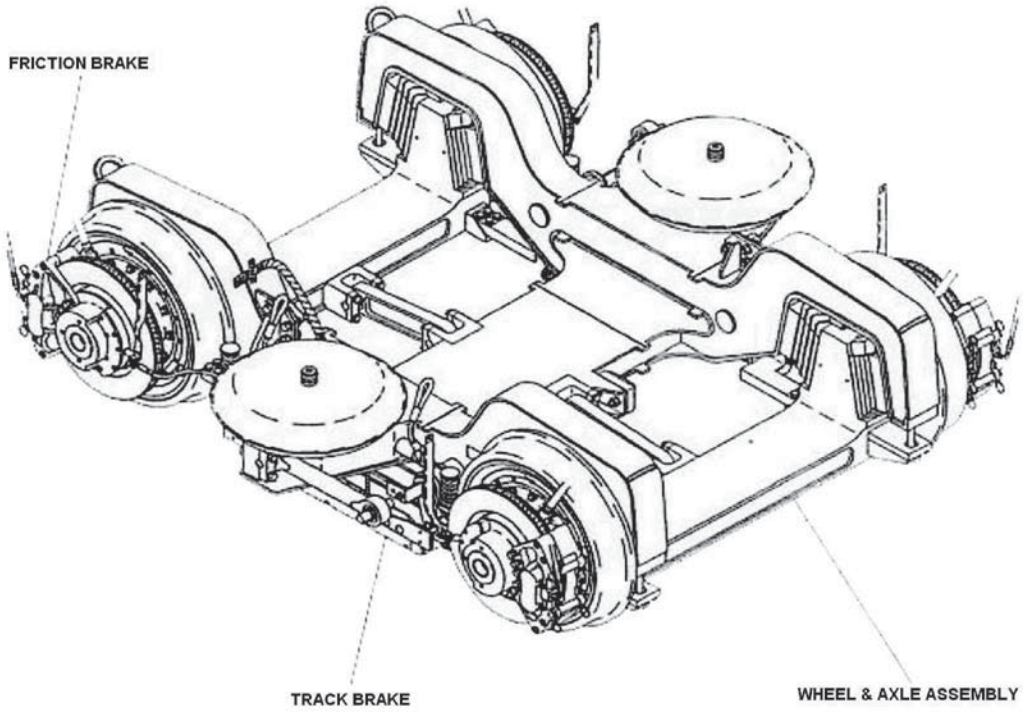

Figure 2.5.4 Kinki Sharyo trailer truck for 70% low-floor LRV

2.5.5.2 Non-Motorized (Trailer) Trucks

Non-motorized trucks are typically located under the articulation joints of LRVs. On low-floor cars, the trailer trucks are located under the center section and don't rotate relative to carbody. They will not have motors and gear units, but will usually have braking systems. Because of their reduced mass, plus the configuration of the LRV carbody with respect to the trucks, the non-powered trucks frequently have lower axle loads than the powered trucks and hence apply less loading to the track. On high-floor cars, they will closely resemble the power trucks with the exception that they typically don't have motors, but the axles rotate, thus promoting steering. On low-floor cars, the non-powered trucks will have appreciably different designs than the powered trucks on the same LRV. In almost all cases of low-floor center section vehicles, there will be no rotating axle and each of the four wheels will rotate independently of the others. **Figure 2.5.4** illustrates a typical trailer truck used under 70% LRVs in several North American cities. It is equipped with the same resilient wheels, primary and secondary suspension, and track brakes as the power trucks on the same cars. Disk friction brakes are located outside the wheels. The wheels are of the independently rotating (IRWs) type and are installed at the end of the low profile crank axle.

Figure 2.5.5 illustrates an axle assembly for a truck with independently rotating wheels. Note the configuration of the cranked axle, permitting the low-floor to pass between the wheels, and the position of the roller bearing races interior to the hub of each wheel.

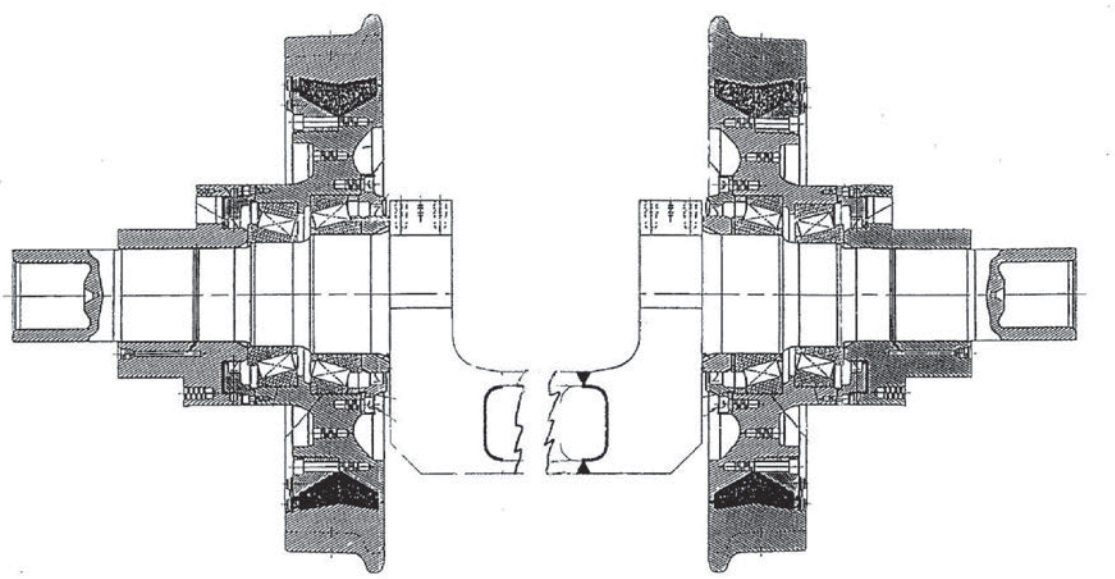

Figure 2.5.5 Kinki Sharyo cranked axle for low-floor LRV trailer truck

2.5.5.3 Load Leveling

Both motorized trucks and trailer trucks typically include air bags as the secondary suspension. Leveling valves installed on the bolster sense changes in pressure between the air bags due to increases or decreases in the passenger loads and automatically inflate or deflate the air bags to restore the car floor level at the predetermined location in compliance with ADAAG.

The adjustment necessary to compensate for the maximum of 1 inch [25 mm] loss of height due to wheel wear is accomplished by shimming under the primary suspension components, typically with rubber chevron springs. The accuracy of this type of adjustment is demonstrated during the vehicle acceptance tests.

The orifice for the air access in the air bag is calibrated to provide the necessary damping precluding resonance. Additional rotary dampers are installed between the bolster and the truck frame.

The carbuilder and the vehicle maintenance organization are largely responsible for ensuring compliance with ADAAG vertical tolerances for matching the elevation of the LRV door thresholds with the station platforms. This includes both the accuracy of car-leveling systems that compensate for variable passenger loading and the periodic insertion of shims in the truck assemblies so as to compensate for wheel tread wear. Vertical rail head wear is typically not accommodated by vehicle shimming as the amount of rail wear can vary significantly from station to station, particularly on a large and mature LRT network. Instead, the track maintainers will be charged with raising the track. Direct fixation track can be shimmed, and ballasted track can be raised. Embedded trackforms usually cannot be raised, and rail replacement might be necessary.

2.5.5.4 Inboard versus Outboard Bearing Trucks

In its simplest form, a truck has two axles that are held parallel to each other by a truck frame. The points at which the frame is supported by the axles are called bearings. Typically, the bearings consist of a box enclosing roller bearing rings inside which the axles rotate. These bearing boxes can be located outboard of the wheels, on extensions of the axles that go beyond the outer face of the wheels, or the bearing boxes can be located inboard of the wheels. The majority of modern LRVs have trucks with inboard bearings, allowing easy access for replacement of the tires on resilient wheels without disassembling the bearings. The overall truck weight is also reduced since the axles are shorter. While outboard bearings are used on some standard gauge truck designs, they are more often found on trucks for tramways using narrow gauge track.

A byproduct of the use of inboard bearings on a conventional solid axle truck is a reversal of the bending moments in the axles compared to an outboard bearings design. With outboard bearings, the moment loading on the axle between a bearing and the adjacent wheel creates tensile forces in the top of the axle and compressive forces in the bottom of the axle. Those forces are counteracted by the weight of the gearboxes, disk brakes, and other axle-mounted equipment so as to somewhat equalize stress in the axle. With inboard bearings, the moments are reversed as are the relative stresses in the axle. However, since the axle is rotating in both cases, these stresses are constantly cycling, setting the stage for possible metal fatigue. In either case, the axles must be designed to accept the stresses from the imposed loads and the cyclic reversal of loadings. However, since the axles are usually the heaviest single element within a conventional truck and since they are largely unsprung mass (with the exception of the minor cushioning provided by resilient wheels), carbuilders have made great efforts to reduce the mass of the axles to the minimum consistent with accepting the service loads within the appropriate factors of safety. Reducing the mass of the axles also reduces the amount of energy necessary to propel the LRV, which can have measurable life cycle cost ramifications. For this reason, many vehicle procurement specifications stipulate a maximum weight for the vehicle and include financial incentive/disincentive clauses for meeting or exceeding the goal.

Where the track design gets into this issue is how the lateral loads from curving are applied to the track by the wheels. With inboard bearings, the lateral forces between the wheels and the outer rail of the curve result in a moment that tends to counteract the other applied moments and actually reduce stress in the axle. A possible problem arises when the track design incorporates restraining rails adjacent to the inside rail of the curve which, by design, share some portion of the lateral load with the outer rail. Any force between the restraining rail and the back of the wheels creates a moment in the wheel and axle assembly that increases the magnitude of the cyclic stresses in the axle. Because of this, many carbuilders and vehicle engineers stipulate that contact should never occur between the back side of a wheel and a restraining rail unless derailment is imminent, such as when the outer wheel has already begun to climb the outer rail. Exacerbating this situation is the fact that some resilient wheels are not designed to effectively transmit lateral forces applied against the back face of the tire.

As discussed in Chapter 4, the use of restraining rail is a recommended practice with a long history of successful use in North America. However, most European track designers make comparatively little use of restraining rails (“check rails” as they are called overseas) and instead

rely on the contact between the outer rail and wheel to accept all curving forces. Therefore, European carbuilders and other international carbuilders schooled in European practice do not typically expect there will be any force acting against the back of the wheel from a restraining rail. BOStrab, the German Federal standard regulations for tramways, actually prohibits routine continuous contact between the back of the wheel and any part of the track structure.

Because of this fundamental difference in design philosophy, if the track design on a project includes restraining rails, that fact must be identified to the vehicle engineers at an early date and clearly explained in the vehicle procurement documents. The carbuilder will likely resist the use of restraining rails since it could require him to use heavier axles, increasing the unsprung mass and overall vehicle weight and possibly triggering a contract disincentive clause. The track engineer must therefore be prepared to strongly defend the use of restraining rails. See Chapter 4, Article 4.3.5, for additional discussion of this issue.

2.5.6 Vehicle Dynamics—Propulsion and Braking Forces

The following parameters establish the maximum forces along the direction of the rails.

The amount of adhesion is the measure of the force generated between the rail and wheel before slipping. A typical 4.8 kilometer per hour per second (3 miles per hour per second) acceleration rate is equivalent to a 15% adhesion level, if all axles are motorized. For a typical LRV with four of six axles motorized, the adhesion rate is 22.5%, which may have some bearing on rail corrugation rate and wear. Increased wear and corrugation rate suggest using hardened rail in acceleration zones and on grades.

2.5.6.1 Tolerances

All acceleration and deceleration values also have tolerances that are due to many factors. The major factors for acceleration tolerance are traction motor tolerances, actual wheel diameter size, and generation and interpretation of master controller commands. This tolerance may range from ±5 to 7%.

All actual deceleration values are dependent on friction coefficients as well as the above issues. The expected tolerance for friction and track brakes should be obtained from the supplier.

2.5.6.2 Maximum Train Size

Acceleration and deceleration forces are applied by all cars in a consist. Therefore, the total rail force per train will depend on the maximum train consist length. If more than one train can be on common rails at one time, this should also be considered. The tractive forces at the wheel/rail contact are independent of the number of cars for self-propelled cars under normal operation. More than one train in a track segment of interest is generally unlikely unless one train was inoperative and being towed or pushed by the other. In that circumstance, the inoperative train would be free-rolling (no power and no brakes) and would hence not apply any tractive effort to the rails. The pushing train might well be up at the limits of adhesion because of the drawbar forces, but that would be no different than the ordinary design criteria. Acceleration/deceleration rates would likely be less for trains with inoperative cars. Slip-slide control will also limit tractive contact forces in non-emergency situations.

2.5.6.3 Load Weight

If the LRV has a load weight function, the acceleration and deceleration forces will be increased at car loadings above AW0 to some maximum loading value. These values should be defined to establish maximum longitudinal track force.

2.5.6.4 Sanding

Car sanders apply sand to the head of the rail in front of the wheel to obtain a higher adhesion coefficient. Sanding in specific locations has a fouling effect on track ballast that should be considered. Sand will also accumulate in flangeways and special trackwork in embedded track. If the wheel/rail interface is over-lubricated—a condition that makes use of sand more likely—the gummy mixture of sand and grease can become a significant housekeeping issue. Sanding may also have a detrimental effect on rail wear.

2.5.6.5 Vehicle Procurement Documents

The procurement documents for light rail vehicles will very often include appendices intended to illustrate the service conditions under which the LRVs must be able to operate. Quite often, this will include plan and profile drawings showing the right-of-way characteristics, including the location of stations, curves, grades, and civil speed limits. If the LRVs are being purchased for an existing route, those parameters will be known exactly. In the case of vehicles for a new LRT line, the preliminary track alignment drawings will often be used as the best available information. The transit agency's manual of design criteria is often also included.

In addition, the vehicle specification will stipulate the required vehicle performance characteristics and conditions under which the vehicle must operate, such as:

- Maximum acceleration, typically 3 mphps [1.34 m/s^2].
- Normal service braking rate (typically the same as maximum acceleration).
- Minimum emergency deceleration, typically 4.5 mphps [2.01 m/s^2] considering a wheel/rail adhesion of 0.5. Higher levels of adhesion may raise the emergency deceleration rate to over 6 mphps [2.68 m/s^2].
- The most demanding service requirements, including routing between terminals, desired schedule speed, distances between station stops, dwell time at stops, passenger loadings, etc.
- Nominal line voltage and maximum line current.

The LRV manufacturer's design team will then determine the equipment and systems necessary for the cars to achieve the specified performance over the route.

2.5.6.6 Braking Forces

Maximum braking forces during deceleration are determined for each track section based on grades and curves and are obtained with a combination of dynamic or regenerative braking (traction motor operating as generator), friction braking, and track brakes—all depending on the available adhesion. A contribution to the longitudinal forces and adhesion controlling is obtained with the load controlling system, sanding system, and slip-slide control system.

The following formula is a sample computation of the longitudinal force (F) on the track created by a three-car train during emergency braking and using a 0.5 adhesion coefficient leading to a deceleration rate (d) of = 3 m/s^2 [6.74 mphps] at an AW3 load of 58,000 kg [about 128,000 pounds] per vehicle.

M = 3 cars x 58,000kg/car =174,000 kg

F = M × d = 174,000/9.81 x 3 = 53,211 kg [117,464 lb]

2.5.7 Dynamic Vertical

Determination of total track force is a complex issue that depends on LRV design features. Typically the vehicle total weight is increased by a factor to include dynamic loading effects. The characteristics of the LRV suspension system should be defined by the manufacturer, who should also provide the dynamic load factor to the track designer.

2.5.7.1 Primary Suspension

Primary suspension provides support and damping between the truck frame and the axle journal bearings. It is the first level of support and vibration control for the bearings above the wheel set.

2.5.7.1.1 Spring Rate

Spring rate is the force per deflection of the coil or chevron primary springs. This relationship may be non-linear for long travel distances. The equivalent vertical, longitudinal, and lateral spring rates will generally be different. Chevron spring suspensions have high longitudinal stiffness, and the solid axles of trucks so equipped turn less easily through curves in response to rolling radius differentials. The longitudinal stiffness should be considered in track curve and rail head profile design.

2.5.7.1.2 Damping

The damping is the “shock absorber” action that provides a force proportional to the velocity of the spring movement. It is designed to minimize oscillation of the springs/mass system at the primary and suspension resonance frequency.

2.5.7.2 Secondary Suspension

Secondary suspension supports the carbody on the truck and controls the range of carbody movement with relation to the truck. The suspension and track alignment basically establish the LRV ride quality. The secondary springs can be either steel coils or air bags.

2.5.7.2.1 Damping

Damping is optimized for ride quality. With an air bag system, orifices in the air supply to the air bags can adjust the damping.

2.5.7.2.2 Yaw Friction

Yaw is the amount of rotation of the truck about a vertical axis with relation to the carbody. With the exception of vehicles that have trucks semi-rigidly attached to a carbody segment (e.g., the Skoda-Inekon streetcar and others), yaw angles as high as 10 to 15 degrees occur routinely along sharply curved track. The truck design and materials used will establish the friction force that restrains truck yaw. High levels of yaw friction contribute to lateral track forces, which can

produce conditions where the wheel climbs over the rail head. The design of related friction surfaces should be such that the friction factor remains constant as service life increases.

2.5.7.3 Maximum Operating Speed

The operating speed limit for all track considers passenger comfort and safety. This criterion should be coordinated with the car design. Civil speed limits for curved track are set by determining the maximum rate of lateral acceleration that passengers can comfortably endure. This is usually in the range of 0.1 g to 0.15 g, which establishes the level of unbalanced superelevation on curves. Speed limits on curves are then established based on the actual and unbalanced superelevation. See Chapter 3, Article 3.2.6, for additional discussion on maximum speeds in curves.

Typically, there are no civil speed limits for tangent track other than arbitrary limits due to the characteristics of the trackway and vehicle. Therefore, the maximum speed on tangent track is typically determined by the vehicle mechanical limits, the train control system, and operating rules. The primary suspension stiffness will determine a stability speed limit that could be quite low.

2.5.7.4 Car Natural Frequency

Light rail vehicles will have a natural frequency that should be considered during the design of civil structures such as bridges or elevated guideways. Trucks and car bodies each have different natural frequencies that should also be considered. Also, car loaded weight affects the carbody's natural frequency. Therefore, the vehicle's natural frequency should be defined at the vehicle's weight extremes, AW0 and AW3. (AW4 is not considered here since it is a theoretical loading only for design of bridges and virtually certain to never be experienced in service.)

If the LRT system already exists and is being extended, there is likely an existing vehicle with natural frequency characteristics that will govern the design of structures. Conversely, if new vehicles are being procured for an existing system, the harmonic characteristics of the existing guideway should be considered in the vehicle procurement specifications. In particular, the bent passage frequency of a car traversing an elevated structure should not be coincident with the car's secondary suspension resonance frequency.

2.6 TRACK GAUGE, WHEEL GAUGE, AND WHEEL CONTOURS

Track gauge, wheel gauge, and wheel contours are some of the most important issues in the relationship between the light rail vehicle and the track. Each of these factors can vary appreciably depending on the characteristics of the light rail system. They are also a dynamic condition due to unavoidable wear of the wheel and rail running surfaces. There are three broad categories in which an LRT system might be placed, each with different ramifications for the track gauge, wheel gauge, and wheel contours:

- An existing or legacy system that has been in operation for many years and already has established standards for gauges and wheels. Presuming that performance is satisfactory, changing any of those parameters should only be undertaken with extreme caution after detailed investigation.

- A new system that will share part or all of its tracks with a freight railroad operation. In such cases, there is usually very little opportunity to change anything, and it may be necessary to default to Association of American Railroads (AAR) and AREMA standards.

- A new system that will be an exclusive operation and have no interaction with freight railroad rolling stock. In this situation, both the trackwork engineer and the vehicle engineer have appreciable latitude to adopt track and wheel gauge and wheel contour standards that can optimize performance and minimize maintenance requirements.

Performance in any of the categories above can be significantly affected by vehicle maintenance issues. If the maintenance plan and budget for the system does not provide for routine wheel truing, the track design may have to accommodate poor curving performance, higher impact forces, and more robust rail support to avoid adverse wear due to poor vehicle maintenance.

2.6.1 Track Gauge

The American Railway Engineering and Maintenance-of-Way Association (AREMA) standard track gauge is established at 56 ½ inches [1,435 millimeters], measured at 5/8 inch [15.9 mm] below the top of rail. While some light rail systems in North America that evolved from legacy streetcar lines use broad gauge track and no small number of European tramways use narrow gauge track, new light rail transit systems worldwide generally adopt standard railroad track gauge. The use of standard gauge track generally facilitates procurement of track materials and track maintenance equipment, although caution is necessary if circumstances result in wheel gauge different than railroad standards. For additional information on track gauge refer to Chapter 4.

2.6.2 Vehicle Wheel Gauge

Vehicle wheel gauge (the distance between defined points on the face of the wheel flange) is always less than track gauge by some freeplay dimension. This is a very important interface issue that must be addressed jointly by vehicle and track designers. Failure to coordinate this issue can lead to interface problems with very costly long-term consequences. This is particularly important if the system will utilize embedded track using groove rails with narrow flangeways. Several LRT systems constructed in the 1980s through 2000 employed AAR standards for wheel contours and gauges, but also employed European groove rails. This resulted in routine interference between the backs of the wheels and the tram of the groove rail, reducing the service life of both.

Standard wheel gauge for railroad cars per AREMA Portfolio Plan basic number 793 is established at 55 $^{11}/_{16}$ inches [1,414.5 millimeters]. However, that dimension, being specified to an arbitrary point on a compound curved surface, is very difficult to measure accurately, particularly as the wheels wear. A more convenient place to measure is between the inside faces of the wheels—a dimension known as the “back-to-back distance,” often abbreviated as “B2B.”

The back-to-back distance for AAR 1B narrow flange wheel sets mounted in accordance with AAR rules is 53 $^{3}/_{8}$ inches [1,355.7 millimeters]. This wheel mounting practice results in $^{13}/_{16}$ inch [20.6 mm] of freeplay between track gauge and wheel gauge. This relatively large dimension is

necessary in railroad work because the acceptable maintenance tolerances for both track and wheel mounting are relatively large.

In contrast, rail transit fleet sizes and track miles are both much smaller than they are for railroads, and it is somewhat easier to achieve tighter maintenance tolerances. In addition, for any rail system operating embedded track in city streets, smaller values of freeplay allow for narrower flangeway widths. Because of these factors, it has long been customary for street railway systems to employ smaller values of track gauge/wheel gauge freeplay than railroads.

The former American Transit Engineering Association (ATEA), which set standards for both streetcar rolling stock and streetcar track in the first half of the 20th century, recommended that freeplay be set at ¼ inch [6.4 mm], which is $^{7}/_{16}$ inch [11.1 mm] less than AAR practice. This reduced freeplay dimension, coupled with the wheel contours recommended by ATEA, resulted in a back-to-back gauge of 54 inches [1372 mm] or more. Legacy systems that still use wheel gauge dimensions based on ATEA practices and any new LRT lines that adopt wheel contours and gauges that differ from AAR practice need to be very careful when procuring new equipment to be certain that their wheel gauge standards are understood by the manufacturers. This is often an issue when procuring maintenance-of-way equipment.

Because of the narrow flangeways provided by most European groove rail sections, LRT systems that employ groove rail in embedded track will generally need to adopt a back-to-back wheel gauge that is wider than the AAR standard. The alternative is to either use one of the few groove rail sections that are specifically designed for use with railroad equipment or to narrow the track gauge to something less than standard. Wide groove rails are generally discouraged because even if they comply with ADAAG maximum dimensions for flangeways they are sufficiently wide that the mobility-impaired and bicycling communities will generally object to their use. Narrowed track gauge may be a practical option in tangent track, but may not be viable in curves and is generally not recommended.

A secondary benefit of narrowed freeplay is reduced amplitude of any truck hunting. However, if conformal wheel contour is also used, a very small amount of movement might still result in a sufficiently large rolling radius differential to initiate self-centering and possibly hunting.

A drawback of smaller values of freeplay between wheel gauge and track gauge is that, assuming tapered wheels, the maximum possible rolling radius differential is reduced. This means that solid axle trucks employing “transit gauge” standard will begin flanging through curves at a higher radius than wheel sets conforming to railroad practice. However, large clearances between wheel and track gauge allows a higher angle of attack at curves, exacerbating flanging. This is not much of an issue on many rail transit lines as their average curve radius is often well below the threshold at which flanging occurs. Track maintenance standards for tight track gauge must be more restrictive, with reduced freeplay, and a minus tolerance of zero is recommended.

Track gauge narrowing has been specifically employed at small radius curves to reduce the angle of attack and thus noise and gauge face wear. In any case, no gauge widening should be employed at any curve on transit systems, as such will promote high angle of attack. While gauge widening is common in the United States, such practice hails from the days of three-axle locomotive trucks.

TCRP Report 71: Track-Related Research—Volume 3: Exothermic Welding of Heavy Electrical Cables to Rail, Applicability of AREMA Track Recommended Practices for Transit Agencies (prepared under TCRP Project D-7) addresses many issues relevant to the interface between LRT track and LRV wheel sets that are not covered by AREMA. It is strongly recommended that the users of this Handbook also consult *TCRP Report 71*.

2.6.3 Wheel Profiles

Wheel profile is one of the most critical vehicle parameters to consider in track design, since the wheel is the primary interface between the vehicle and the track structure. The wheel profile must be compatible with the rail section(s); the special trackwork components, including switch points and frog flangeways or moveable point sections; the guard rail positions to protect special trackwork components; and restraining rail if used on sharp radius curves.

Once accepted, any changes to the wheel profile (especially tread and flange width) must be evaluated by both vehicle and track designers. In more than one instance, the wheel profile has been altered at the last minute by the vehicle side of a project without informing the track designer, resulting in unsatisfactory performance of both the track and vehicle.

The first edition of the *Track Design Handbook for Light Rail Transit* (also known as *TCRP Report 57*) illustrated a dozen different wheel contours that were in use on North American light rail lines at the time. The differences were startling, and there was seemingly no consistency. Several designs had their origins in AAR practice, while others could be traced back to ATEA designs. Still others resembled wheels used on some European railway systems, and their selection may have been influenced by the overseas suppliers of the LRVs and/or track materials. Looking at those wheel designs in light of current understanding of rail/wheel mechanics, only two or three have sufficient merit to warrant consideration for any new light rail rolling stock. Rather than possibly misleading readers into thinking all those wheel designs are all recommended designs, they have been omitted from this second edition in favor of discussions of characteristics that can be found in a good wheel design. Parties with an interest in some of these other wheel contours can consult *TCRP Report 57* for additional information, although it must be understood that some systems may have changed their wheel contour since *TCRP Report 57* was published.

2.6.3.1 AAR-1B Wheel Contour

The Association of American Railroads (AAR) promulgates two standards for wheel contours on rolling stock. The AAR-1B wide flange contour is generally of no interest to transit work. The AAR-1B narrow flange contour is used on locomotives, railroad passenger cars, and some freight equipment. Both versions of the AAR 1B wheel were adopted as their standards during the 1990s, replacing much older designs that had been AAR's standards since the 1920s.

AAR-1B wheels incorporate a compound curve radius at the throat between the flange and the wheel tread. This is designed to conform to similar radii on the heads of AREMA standard rail sections. This conformal contact facilitates curving by maximizing the rolling radius differential between wheels on the same axle and also promotes self-centering of wheel sets in tangent track. The conformal contact at curves may also reduce contact stresses and thus wear. The AAR's former wheel design, which is still used by several LRT systems, has a single radius in the throat. The wheel profile is considered to be conformed to the rail profile if the gap between the

wheel and rail profile is less than 0.5 millimeters [0.02 inches] at the center of the rail (in single-point contact) or at the gauge corner (in two-point contact). Both the old and current AAR wheel designs incorporate a 1:20 taper on the wheel tread so as to facilitate truck centering on tangent track and self steering on slight curves.

The AAR-1B wheel profile is an evolution from a design first proposed by Professor Herman Heumann (1878–1967), a German railway engineer who did pioneering work in the field of wheel-rail contact mechanics. Some elements of Professor Heumann's work have been superseded by subsequent research (notably his endorsement of a 70-degree flange angle), but that is the result of better analytical methods and changes in the demands placed on the rail wheel interface rather than any flaws in his theories.

Tests by the AAR at the Transportation Test Center in Pueblo, Colorado, have shown that the AAR-1B wheel profile provides

- A lower lateral-over-vertical (L/V) load ratio in a 764-foot [233-meter] radius curve than the previous AAR non-conformal wheel.
- A lower rolling resistance than the previous AAR profile. Arguably, this is less important in a transit vehicle, which might have 66% or even 100% of its axles powered, versus a locomotive-hauled freight train, which might have only 5% of the axles powered, but it does have some ramifications for life cycle energy and maintenance costs.
- Lower critical hunting speeds than the old AAR wheel profile. This means that, all other things being equal, trucks equipped with the AAR-1B wheel will commence hunting at a lower speed than the AAR's old non-conformal wheel. The hunting speed is primarily a function of wheel tread taper at the center of the tread running surface.

The last bullet point is significant, and some discussion is appropriate. "Hunting" is the tendency of a wheel set with tapered wheels to uncontrollably oscillate from flange to flange while seeking to center on the track with a consistent rolling radius on each wheel. This is a dynamic condition, highly sensitive to the natural frequency of the truck design as well as the presence or absence of dampers (e.g., shock absorbers) to control truck rotation (yaw). With a conformal wheel, compared to a wheel having either a straight taper leading to a small flange/tread radius (or even no taper in the case of a cylindrical wheel), a smaller amount of lateral movement is required to create an appreciable difference in rolling radius, thereby initiating self-centering action. Overcompensation could then initiate hunting behavior at certain speeds.

Informal observations suggest that "worn wheel" designs similar to the AAR-1B—which was designed for relatively large values of gauge freeplay per freight railroad standards—may on some vehicles and truck designs hunt excessively when freeplay is tightened down to transit standards. This is likely due to running closer to the flange throat, where the taper becomes large. The overall system needs to be proportioned so that with the wheel set centered on tangent track there will be no routine contact between the gauge corner radius in the wheel flange throat and the crown radius of the rail head. This is an area that requires additional research.

Wheel tread wear will tend to reduce the taper from the new condition. In the extreme case, when maintenance intervals are too long or wheel truing is simply non-existent, excessive wear of the wheel will produce a "false flange"—a relatively unworn zone on the outside of the wheel

tread that lies below the plane of the top of rail. On the field side of the concave worn tread, the wheel taper will actually be negative. Such worn wheels are often referred to as having a "hollow tread profile." Poor curving performance will occur, with potentially poor performance on tangents, contributing to rail corrugation and wear.

2.6.3.2 Transit Wheel Design and Selection

While shared track with a freight railroad operation might force the selection of the AAR-1B narrow flange wheel and AAR wheel gauge, most new LRT operations have more latitude in selecting an optimal wheel profile.

Rail car designers have several computer programs available that enable them to model the dynamic characteristics of the vehicle, including the behavior of the proposed wheel profile for a given trackform and variations in rail head shape, gauge freeplay, and other factors. Examples include NUCARS, AdamsRail, and VAMPIRE.

Figure 2.6.1 illustrates a wheel contour that has been successfully employed on a U.S. LRT system that uses both 115RE tee rail and 51R1 groove rail. It could be considered as a starting point for determination of the optimal wheel for a new LRT system without railroad interface. Figure 4.2.2 in Chapter 4 illustrates the same wheel superimposed on the track and illustrating gauge and freeplay issues. Since the time when this wheel was developed, the dimensions of 115RE rail have been revised to incorporate an 8-inch [300 mm] crown radius, hence this wheel profile may no longer be optimal.

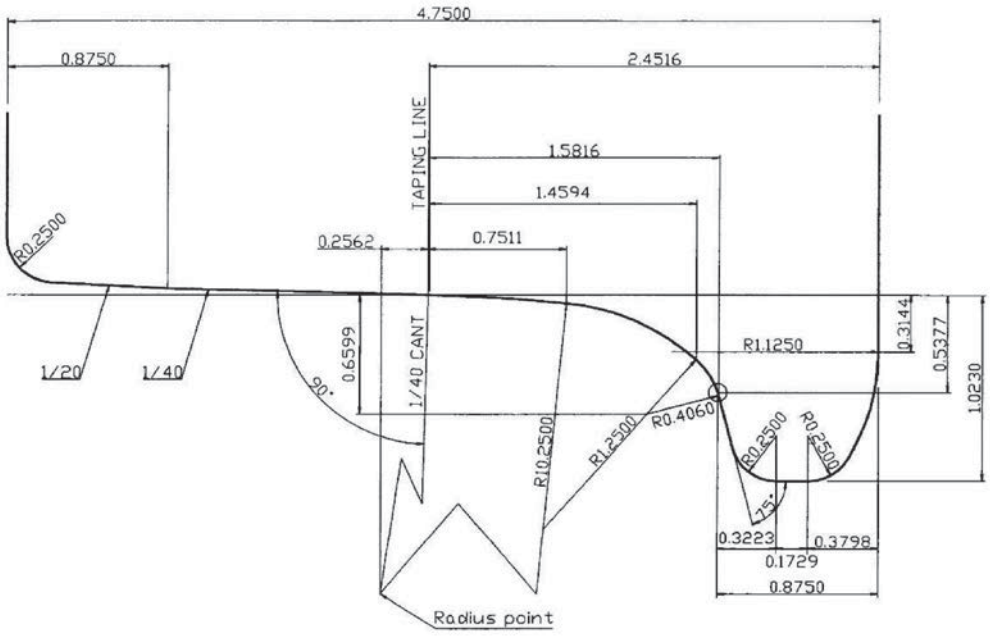

Figure 2.6.1 Candidate initial LRV wheel profile

(All dimensions in inches)

The paragraphs that follow describe some of the issues that must be considered when selecting or developing a wheel profile for light rail transit.

2.6.3.2.1 Tread Conicity

Wheel treads virtually always have a conical taper when new (usually 1:20) so as to promote self-centering in tangent track and some degree of steering in flat curves. Conical/tapered wheels have been common since the early 20th century. However, a very few legacy rail transit properties continue to use cylindrical wheels, having originally adopted them long ago to resolve problems with uncontrolled truck hunting. That solution came with the penalty of loss of self-centering and increased wear on rails and wheel flanges in curves. Cylindrical wheels also need more frequent maintenance to correct the development of false flanges. Better methods are available to control hunting today through truck design, so cylindrical wheels are not recommended.

Some transit properties have adopted flatter or steeper tapers than 1:20 and/or use a steeper conicity outboard of the normal wheel/contact zone. The latter defers the need to do wheel truing to correct hollowing of the wheel tread, but, in general, frequent wheel truing is strongly recommended as part of a comprehensive preventative maintenance program. Some literature suggests that tapered wheels may promote wheel squeal at curve, due to a positive feedback effect as the wheel vibrates across the rail head. This behavior is theoretical, but may explain why wheel squeal appears to be more prevalent at rigid track than in poorly maintained track built with jointed rail that is only loosely fastened to the ties. This is a curious situation that deserves more investigation.

2.6.3.2.2 Tread Width

The tread on AAR wheels is over 4 inches wide, that being necessary to ensure the wheel can reliably bridge the open throat of the intersecting flangeways in turnout frogs, given the relatively loose tolerances on railroad track gauge and wheel set maintenance. Transit systems, having a captive fleet and higher standards for track and wheel set maintenance, can generally employ narrower flangeways in frogs and proportionally narrower wheels. If the track system employs flange-bearing frogs throughout, the wheel tread can be very narrow as the wheel tread is not in contact with the frog through the open throat. Narrow wheel treads also reduce the unsprung mass of the wheels, with appreciable benefits concerning impact forces and energy consumption. Narrow tread wheels are typically combined with wider back-to-back wheel gauge, the reduced freeplay compensating for what might otherwise be a reduction in the available wheel/rail contact surfaces. See Article 2.6.6 for additional discussion on wheel tread width.

2.6.3.2.3 Flange Face Angle

Older wheel designs, such as those recommended by the former ATEA, had relatively flat flange angles. An angle of 27 degrees to the vertical (63 degrees to the axle) was common. Research at the Transportation Technology Center, Inc. (TTCI), as documented in *TCRP Report 71: Track-Related Research—Volume 5: Flange Climb Derailment Criteria and Wheel/Rail Profile Management and Maintenance Guidelines for Transit Operations*,[15] demonstrated with numerical simulations that wheel flanges positioned at an angle of 72 to 75 degrees with respect to the axle are much less likely to climb the rail than the old flatter flange angles. The factor that describes the propensity for a wheel to climb the rail is known as the Nadal Value. Wheels that comply with the old ATEA designs were found to have Nadal Values of about 0.70 to 0.75. By contrast, the

AAR-1B wheel and transit wheels of similar design have Nadal Values of about 1.1, indicating a much reduced tendency to climb the rail and hence a greater margin of safety against derailment.

To be fully effective, the 75-degree flange angle should be constant (i.e., not part of a curved surface) for a distance not less than 0.1 inch [2.5 mm]. APTA adopted this standard as part of their recommended practice for commuter railroad equipment.[11] As of 2010, APTA had not endorsed this feature for light rail and metro rail passenger equipment, but it can be safely asserted that it represents good practice. Many European tramways use wheels that have an even steeper flange face angle of 1:6 (about 80.5 degrees to the axle), which matches the gauge face slope that is common on European groove rail sections.

2.6.3.2.4 Flange/Tread Radius

As noted above, nearly all modern wheels incorporate a conformal compound curve radius in the throat between the wheel tread and the flange. This should closely match the radii used on the gauge corners of the rails to be used on the LRT. Designers are cautioned against mixing different rail sections in the track design unless the selected sections present a reasonably consistent contact surface to the wheel. In that regard, it should be noted that many groove rail sections have gauge corner radii that are radically different from that of 115RE tee rail.

2.6.3.2.5 Flange Back Angle/Radius

Most wheels, including the AAR-1B, have a relatively broad radius between the radius on the flange tip and the flat face of the back of the wheel. This eases the transition of the wheels into guarded special trackwork and is hence desirable for smooth operation. In the case of track systems that employ restraining rail, the angle of the back of the wheel should be carefully considered with respect to both the horizontal angle of attack between the wheel and the restraining rail and the vertical angle of the restraining rail. Three dimensional modeling of the contacting surfaces is suggested.

2.6.3.2.6 Flange Height

The flange height is the vertical distance from the tip of the flange to a point on the wheel tread known as the taping line (see Article 2.6.3.2.9). Legacy streetcar lines, particularly those with flange-bearing special trackwork, often use very short wheel flanges. Three-quarters of an inch [19 mm] is common, which contrasts sharply with AAR wheel flanges that are 1 inch [25 mm] tall. Short flanges have several serious design issues:

- They are generally incompatible with the AREMA 5100 undercut switch point design because the tip of the wheel flange is above the top of the leading end of the switch point. On one LRT project, short flanges on legacy rolling stock that had worn even shorter in service would routinely climb the second cut on the top of the diverging switch point and derail. An aggressive wheel reprofiling program along with a wholesale modification of the stock rails was necessary to stabilize the situation.
- Their short height also provides a very narrow contact band with the gauge side of the rail when passing through curves, leading to accelerated gauge face wear on both the rails and the wheels.
- They provide virtually no height for the desirable minimum straight flange face angle when combined with a conformal compound radius in between the flange and the tread.

For these reasons, the recommended minimum flange height is 1.0 inch [25 mm].

2.6.3.2.7 Flange Thickness

Typically, the flange thickness—the horizontal dimension from the projected vertical back face of the wheel to the gauging point on the front of the flange—should be about $^{7}/_{8}$ to 1 inch [22 to 25 mm]. This allows for a reasonable amount of flange face wear before wheel truing becomes essential. In general, wheel truing should not be deferred until the flange thickness reaches a condemning limit, since by then it might not be possible to restore the flange without removing an excessive amount of the wheel tread surface, substantially reducing the wheel diameter. Reduction of wheel diameter often triggers the need to shim the trucks so that the vertical relationship between the vehicle doorways and the platform remains in compliance with ADAAG.

If the track design will use groove rails with extremely narrow flangeways (generally any flangeway less than about 1 ½ inches [38 mm] wide), it will usually be necessary to reduce the flange thickness from the recommended dimension above. Such thin flanges will require more frequent wheel truing and are not recommended.

2.6.3.2.8 Flange Tip Shape

The tips of the wheel flanges on systems that use flange-bearing special trackwork tend to wear flat or nearly so, slightly decreasing the height of the flange. To prevent this loss of height, the wheel flange for use with flange-bearing frogs should have a tip that is either flat or has a very broad radius for a width of at least ¼ inch [6 mm] to reduce contact stresses. This then compounds into a shorter radius that blends into the angles on the front and back face of the flange.

2.6.3.2.9 Wheel Diameter

LRV wheels are generally 24 to 28 inches [610 to 710 mm] in diameter. This measurement is made at a point on the tread that is a consistent distance from the back face of the wheel and nominally where the wheel tread contacts the top of the rail when the wheel set is centered on the track. It is known as the “taping line” since that is the location where the circumference of the wheel is measured with a specially calibrated tape.

The diameter of a wheel has a direct effect on the length of the “footprint” that the flange has at the top-of-rail level. This in turn affects how the wheel interacts with the rail, especially in curves and through special trackwork. The footprint of small diameter wheels could be less than the length of open frog throats and could present challenges with respect to providing proper guarding of the frog. See Chapter 4 for a discussion about the generation of Wharton diagrams and Nytram plots and for the determination of the most appropriate track gauge and flangeway widths for a given wheel. Mixed fleets that have more than one wheel diameter must consider each one independently, even if they all have the same wheel profile.

2.6.3.3 Independently Rotating Wheels (IRWs)

Independently rotating wheels, having no solid axle to force paired wheels to have the same rotational velocity, behave appreciably differently in curved track. Curving behavior is modified, reducing longitudinal slip, but flange face wear is greater on IRWs than on the wheels of the power trucks on the same LRVs due to increased angle of attack. IRWs tend to produce more flanging noise than solid axle wheel sets, again due to increased angle of attack and lateral creep velocity. This issue was investigated in TCRP Project C-16, and informal observations that had

been made on several transit properties operating 70% low-floor cars were confirmed.[16] As a result of this accelerated wear, it is generally necessary to reprofile IRWs more frequently and replace the resilient wheel tires more often than on solid axle wheel sets.

2.6.3.4 Miscellaneous Considerations for Wheel Contours

2.6.3.4.1 Historic Streetcars

Several light rail transit systems have antique streetcars (or modern replicas of same) that are operated over the tracks of the system on either an occasional or scheduled basis. The wheels on such rolling stock must be considered to the same degree as those of the LRV fleet. In general, any such vehicles should be retrofitted with wheel contours conforming to the adopted standard for the system. Exceptions might be made for a one-time use, such as the opening day ceremonies for a new LRT system, provided the wheels on the heritage vehicle are in good condition and the back-to-back wheel gauge is consistent with the special trackwork. Badly worn wheels, particularly any which have short flanges or false flanges, should not be permitted

Even if the heritage vehicles will be equipped with new wheels, some modifications may still be required in the event that the heritage vehicles have wheel diameters or truck wheelbases that are substantially different from the regular LRV operating fleet. Many pre-PCC vintage streetcars have wheel diameters that are appreciably different (both much larger and much smaller) than those of modern LRVs. These differences directly affect the footprint of the wheel flange at the top of rail elevation. Such wheels should be evaluated closely using Filkins-Wharton diagrams and the Nytram plots as discussed in Chapter 4.

2.6.3.4.2 Shared Trackage with Freight Railroad

In the event that the LRT shares track with freight trains, special trackwork that conforms to AREMA standards for flangeways and check gauge and adoption of the AAR-1B wheel (or something close to it) will usually be essential.

However, if the LRT system also includes embedded track sections using narrow flangeway groove rails, it may be necessary to both employ a compromise wheel contour and modify the special trackwork in the shared-use area. Such combined systems became popular in Europe during the 2000s, following the success of a pioneering “tram-train” operation in Karlsruhe, Germany. Such systems typically use ordinary tramway tracks in downtown areas and switch onto local or regional freight railroad tracks in suburban or interurban areas. Compatibility is achieved by both using a modified wheel, as seen in **Figure 2.6.2**, and providing elevated guard rails opposite frogs in the shared track areas.

In Figure 2.6.2, the 7.5 mm [0.30 inch] projection on the back face of the wheel provides a back-to-back distance that complies with European practice on freight railways while the back-to-back gauge at the wheel tread elevation complies with transit practice. The overall width of the wheel provides for safe operation over railroad frogs while the outer taper provides assurance that the wheel tread overhang will not initially contact the pavement in groove rail areas. (Some contact may occur as the rail wears and would need to be corrected by pavement grinding.) Wheel gauge and gauge freeplay match transit practice and present no problem on well-maintained freight track.

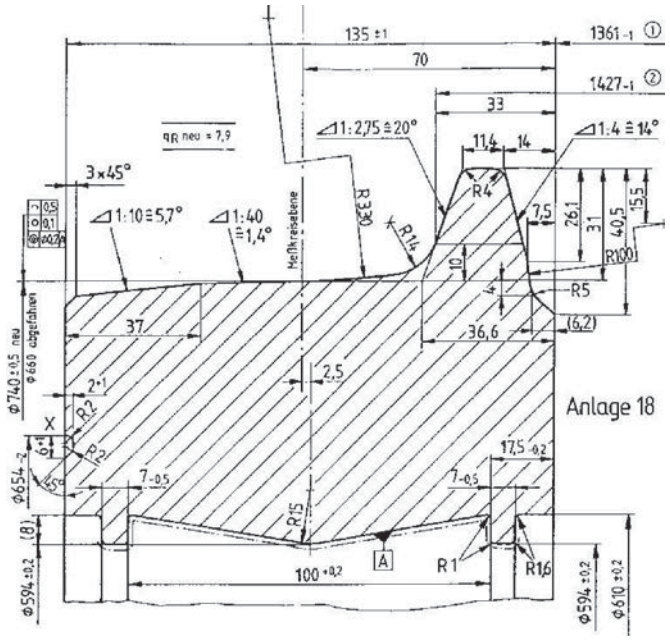

Figure 2.6.2 Compromise wheel for Karlsruhe tram-train

(all dimensions in millimeters)

No tram-train systems have been constructed in North America, although DMU operations in southern New Jersey and Austin, Texas, have some tram-train characteristics. There are institutional issues related to the regulations of the Federal Railroad Administration that make it somewhat unlikely that tram-train technology can be fully applied in the United States. That situation notwithstanding, the Karlsruhe wheel is illustrative of what can be possible when trackwork and vehicle designers collaborate to achieve a desired goal.

2.6.3.5 Average Worn Wheel Conditions

Chapter 2 of the first edition of the *Track Design Handbook for Light Rail Transit* included an extensive discussion of investigations made concerning interactions between trackwork and badly worn, "hollowed" wheels with pronounced "false flanges" on the outer edges of the wheels. That discussion originated in research done for freight rail operations and generally has no applicability to a light rail transit system that performs routine wheel truing as part of a comprehensive preventative maintenance program.

The focus of investigations into wheel/rail interactions is generally on the performance of new wheels running on new rail, a condition that exists only briefly on any project. Arguably, the condition of most interest is the behavior of the system with both rail and wheels "worn in," but well before either reaches a condemning limit. Wheels generally wear much faster than rails. So some investigation about the performance of average worn wheels running on average worn rails might be appropriate. For an operating system with little maintenance budget, the track designer may be faced with accommodating a variation of tread profiles for the same vehicle. All of these options are appropriate for wheels and rails in good condition as well. Designing for the worst-case profile is appropriate, and close coordination between track and vehicle maintenance providers is necessary in any case.

2.6.4 Maintenance of the Wheel/Rail Interface

When the first edition of the *Track Design Handbook for Light Rail Transit* was published, there had been relatively little investigation into the rail/wheel interaction under transit vehicle loadings. Since that time, there has been a good deal of investigation under the auspices of TCRP Project D-7, with the results published as a series of volumes collectively known as *TCRP Report 71*. As of 2011, the D-7 project is ongoing (and is expected to continue indefinitely), providing factual information specifically targeted at rail transit instead of conjectural extrapolations of the results of research done under freight railroad loading.

Rail transit system maintenance procedures have come under increased scrutiny since 2000. As of this writing, the states are responsible for oversight of the process,[17] but federal oversight is increasing. Partially in response to this regulatory scrutiny, APTA has developed recommended practices for transit rail car maintenance, including wheels.[11], [12] Most rail transit systems are now following system-specific wheel management procedures, consistent with the APTA guidelines, with respect to inspection and maintenance of wheels including truing of worn wheels. New Jersey Transit has developed comprehensive standards for wheel maintenance that could be considered a model program. This program includes the following standards:

- Wheel maintenance procedures are included in the System Safety Program as a mandatory requirement.
- Wheel wear conditions are checked with either a digital output hand-held profile gauge or on the truing machine as part of a mandatory daily vehicle inspection.
- Wheel reprofiling is performed either at fixed intervals—every 30,000 to 40,000 miles [48280 to 64374 km] depending on the truck design—or as periodic measurements indicate the need for corrective action.
- Intermediate wheel profiles are used as determined by software incorporated in the wheel truing machines. As many as 20 variants of corrective actions are recommended by the machine so as to minimize the removal of metal from the wheels.

With this program in place, New Jersey Transit has increased resilient wheel tire life dramatically, typically achieving 200,000 to 250,000 miles [322,000 to 402,000 km] of service before tire replacement is necessary.

Maintenance of the track side of the wheel/rail interface, principally through a comprehensive program of rail grinding and strategic lubrication, is equally important. See Chapters 9 and 14 for discussions of these topics.

2.6.5 Matching Wheel and Rail Profiles

Since wheels are a machined item and finished on a lathe, it is relatively easy to procure customized wheel contour designs to suit particular applications. The same flexibility is not available in the selection of rail profiles since rails are finished on a rolling mill. Further, of the roughly two dozen rail sections commonly available, only a very few are actually suitable for use by rail transit. However, wheel and rail profiles must be compatible, which generally means that the wheel profile needs to be detailed to conform to the as-rolled head profile of the selected rail.

As with wheel profiles, the majority of the research and development work regarding rail head profiles and rail profile grinding has been undertaken by and for the railroad industry. While the transit industry can also benefit from this research, readers are cautioned that recommendations for heavy haul railroads are very often less than entirely applicable to the transit industry. The difference in maximum wheel load between a light rail vehicle and a fully loaded freight car can be a factor of 4 or 5. Because of this large difference, rails used in transit service will not be subjected to wheel forces of the magnitude exerted by freight cars. Therefore, theories of rail gauge corner fatigue, high L/V ratios, and the threat of rail rollover that pertain to freight railroads are generally less applicable on a transit system.[18]

To illustrate the differences between conformal and non-conformal wheels, **Figure 2.6.3** illustrates the 115 RE rail section used on contemporary LRT systems in conjunction with both the obsolete AAR wheel profile and the newer AAR-1B wheel profile. Note how the non-conformal two-point contact wheel/rail relationship of the non-conformal wheel transfers the vertical load from the gauge corner toward the centerline of the rail. This combination reduces the wheel radius at the contact location, which is detrimental to steering and introduces accelerated gauge face wear. In practice, the wheel gauge corner will tend to wear to the rail and vice versa, developing some modest conformal contact over the long term. However, as the system matures, normal maintenance will result in the introduction of new and freshly reprofiled wheels and replacement of worn rail with new rail, resulting in inconsistent wheel/rail contact. A mixture of rails and/or rail cant conditions on a single system will result in non-uniform rail profiles at the gauge corner and tend to frustrate achieving a systemwide stable gauge corner profile for the worn wheel.

To improve wheel/rail interface contact on older systems, alternate wheel shapes may be considered. During the early design stage of new transit systems, transit wheel profiles should be considered that match or conform to the rail section(s) to be used on the system. In the process of wheel design, the design engineer must consider both the rail section(s) and the rail cant at which they will be fastened. For additional information on rail sections, refer to Chapter 5 of this Handbook. For additional information on rail cant selection and benefits, refer to Chapter 4, Article 4.2.5.

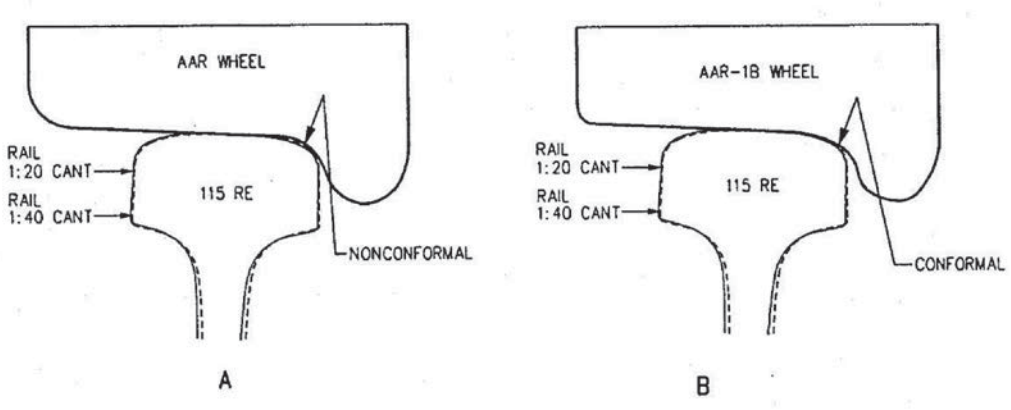

Figure 2.6.3 Wheel-rail interface

2.6.6 Wheel Tread Widths and Flangeways at Frogs

When a wheel passes through a frog, the wheel tread must pass over the open throat of the intersecting flangeway. In an ordinary (not flange-bearing) frog, the load on the wheel will briefly transfer from the inner to the outer part of the wheel tread and then back again as the wheel passes over this gap. For this transfer to be smooth, the wheel tread must be appreciably wider than is required to support the wheel in ordinary track. See Chapter 6, Figures 6.6.1 and 6.6.2 for an illustration of how a wheel traverses a frog.

The large value of freeplay between AAR wheel gauge and standard track gauge requires a wider flangeway opening through frogs and guard rail flangeways than when following transit standards. The wider flangeways allow larger lateral wheel movement, resulting in less wheel tread contact if the wheel set has shifted furthest from the gauge face of a frog point. If the wheel tread is too narrow, this condition results in hammering of the wing rail and the frog point due to insufficient tread support when the wheel transfers between the two components. Narrow wheels traversing the frog in a facing point direction lose the wing rail wheel support too early, resulting in premature transfer of wheel load to the narrowest portion of the frog point, resulting in batter and crushing of the frog point. In a trailing point orientation, the batter occurs on the wing rail instead of the frog point. To minimize these problems, the AAR standard wheel has an overall width of $5\,{}^{23}/_{32}$ inches [145.3 mm].

A wider wheel tread increases the weight of the wheel, thereby increasing the unsprung mass of the truck and impact forces by a small but measurable amount. Wide wheels can also abrade adjoining pavement in embedded track areas. A narrower overall wheel width is therefore desirable. The suggested minimum width for a transit system that shares its track with freight cars and hence needs to follow AREMA-recommended practices for flangeway widths, is 5 ¼ inches [133 millimeters]. This dimension includes a ¼-inch [6-millimeter] radius at the field side of the wheel tread. Wheels that are narrower cannot be used with railroad standard flangeways and wheel gauges as doing so will lead to improper wheel traverse through special trackwork components. Reduction of both flangeway widths and wheel widths is possible in special trackwork that does not need to deal with freight equipment, particularly if transit gauge freeplay standards are followed.

2.7 RESILIENT WHEELS

Nearly all North American LRVs use resilient wheels such as the Bochum Bo54, Bochum Bo84, SAB, and the Acousta-Flex wheel designs. A few other designs are also in use.

Resilient wheels have a long history of use on rail vehicles as a means of reducing the impacts between the rail and the vehicle. The earliest resilient wheels actually appeared in the late 19th century, using compressed paper as the cushioning element in the wheels beneath railroad sleeping cars. Several experimental designs of resilient wheels existed for streetcars in the 1920s, but the first large-scale use of cushioned wheels occurred with the introduction of the PCC streetcar in the mid-1930s. The PCC resilient wheels (there were several variations) were of the “sandwich” design, with the compressed rubber components oriented in the plane of the wheel and hence in shear under loading. Such wheels could handle a maximum vertical

wheel load of approximately 6000 pounds [about 2700 kg], which was sufficient under the relatively light PCC car.

Heavier cars required more robust resilient wheel designs than could be managed with a sandwich design. One of the more popular designs was the Bochum Bo54 wheel, introduced in 1954, which placed a series of rubber blocks in compression between a wheel hub and outer ring-shaped tire. The Bo54 design worked well, but required sophisticated equipment ("The Bochum Press") to change the tires. In response to that issue, the Bochum Bo84 design made tire replacement much easier and cost-effective. Bo84 resilient wheels were designed to withstand a vertical wheel load of 12,000 pounds [5,443 kg]. Other designs based on the same principles are available from international vendors, many of whom have licensed U.S. firms to manufacture their products.

Ignoring heritage streetcars, there are extremely few light rail vehicles that still utilize solid wheels. The advantages of resilient wheels compared with solid steel wheels are

- Noise reduction/attenuation due to the rubber's absorption of structure-borne vibrations. One study revealed a reduction of noise of 25 to 30 dBA for resilient wheels versus solid wheels. Resilient wheels are particularly effective in reducing sustained wheel squeal at curves, probably due to damping and the ability of the tire to deflect about a vertical axis through the contact patch. However, flanging noise is not reduced, though it is generally of much lower amplitude than sustained wheel squeal from solid wheels.
- Decrease of wheel and track wear due to the rubber blocks placed between the tire and the hub. One study suggests that flange face wear is half what it would be for solid wheels. This has distinct advantages with respect to wheel truing since, when wheels are turned, most of the reduction in wheel diameter is not to remove defects in the wheel tread but rather to restore the thickness of the wheel flange.
- Reduction of unsprung mass to the weight of the tire. By contrast, in a truck with solid steel wheels, the entire mass of the wheels and axle is unsprung.
- Resilient wheel tires are available with better material properties than those of rigid wheels. The typical resilient wheel tire has a hardness of 320 to 360 BHN compared with solid wheels, which have a hardness of 255 to 290 BHN. The harder wheel is hence closer to the strength of heat-treated premium rail. Softer wheels would have been sacrificial to the rail when it comes to wear. The harder wheels are closer to parity.
- Reduced wheel set shock and vibration, which is beneficial to trucks with rigid couplings between the axe and gear box out shaft. Brake discs mounted on the axle also benefit from reduced shock and vibration.

The rubber springs of both the Bo84 wheel and Bo54 wheel are mounted in compression for vertical loads and act in shear for lateral loads. The lateral stiffness of the Bo54 and Bo84 wheels is controlled by providing a chevron-shaped cross section, which is incorporated into the Bo84 wheel as shown in **Figure 2.7.1**. Lateral shift of the tire relative to the hub of the wheel is thereby significantly reduced. Modern resilient wheel designs have also increased the allowable tread wear, and tire replacement can now be performed without truck removal. Higher loadings are

now possible without overstressing the wheel. One vendor reports commonly handling lateral forces of up to 45 kN [10,000 lb] with a vertical load of 60kN [13,500 lb] with no reported failures or problems.

Figure 2.7.1 illustrates Bo84 wheels as used by New Jersey Transit.[9] The larger wheel tire on the left uses an AAR-1B wheel profile as well as AAR back-to-back wheel gauge and freeplay and is used on NJT's Hudson-Bergen LRT line. The smaller wheel is used on NJT's Newark City Subway routes and accommodates a back-to-back wheel gauge of 54.125 inches [1375 mm] and a reduced value of freeplay. While the same light rail vehicle is used on both routes, a different wheel is required on the Newark City Subway routes because they evolved from a legacy streetcar system.

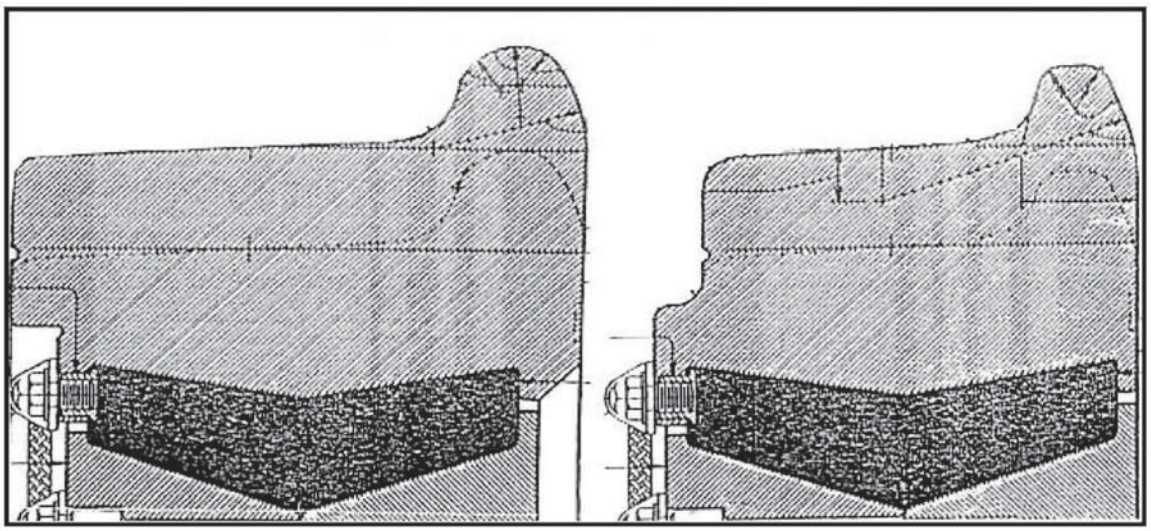

Figure 2.7.1 Bo84 wheels used by NJ Transit

For additional information on resilient wheels, see Chapter 9, Article 9.3.3.9.

2.8 ON-BOARD VEHICLE WHEEL/RAIL LUBRICATION

As is discussed in Chapter 9 of this Handbook, lubrication of the wheel-rail interface is a proven method of reducing wheel squeal noise. A simple observation of this can be made on any rainy day, when merely a thin film of water dramatically reduces wheel squeal. Traditionally, the application of lubricants and friction modifiers to the rails has been a responsibility of the track maintenance department. However, maintenance of trackside lubrication equipment has always been difficult and proper operation therefore erratic. Common problems include either too much or too little product applied and too little of it finding its way to the point of need. In addition, application of friction modifiers in embedded track areas can cause safety issues with motor vehicle traffic and pedestrians.

Because of these issues, placing the lubrication equipment on the light rail vehicle is very attractive. It brings the equipment to the vehicle maintainer for servicing instead of requiring the track maintainer to go to multiple equipment sites, making maintenance and resupply more likely to occur. It also provides an opportunity to better control the application rate. However, the

initial method of on-board lubrication, solid stick lubricators held by spring pressure against the flange of the wheel, have generally been unsatisfactory.

Several situations have changed that collectively show promise of creating an optimal method of getting friction modifiers to the locations that most need it:

- Better lubricants and friction modifiers that are vastly superior to and more environmentally friendly than common mineral oils and greases. These products have better characteristics for friction values, adhesive power, corrosion protection, and phase separation. They are also stable independent of temperature and can be sprayed. See Chapter 9 for additional information.

- Reliable spray equipment designed to match these new products that can be mounted on light rail vehicles.

- Global positioning system (GPS) technology that enables automatic activation of the on-board equipment at curves and other locations requiring the friction modifier without demanding action by the vehicle operator.

As of 2010, approximately a half-dozen rail transit agencies in North America have adopted on-board spray equipment for targeted application of wheel flange and top-of-rail friction modifiers. This system shows both good results (such as control of wheel squeal to less than 80 dBA) and great promise for being a maintainable technology.

2.9 VEHICLES AND STATIONS—ADA REQUIREMENTS

The Americans with Disabilities Act (ADA) requires that public operators of light rail transit systems make their transportation services, facilities, and communication systems accessible to persons with disabilities. New vehicles and construction of facilities must provide the needed accessibility in accordance with the ADA Accessibility Guidelines (ADAAG).

As a guideline, new light rail transit stations should be designed taking into consideration the ultimate ADA goal of providing universal access for persons with disabilities. The track alignment designer may need to consider the following when setting the track horizontal and vertical alignment.

- Horizontally, the ADAAG requires providing platform edges that are within 3 inches [75 millimeters] of the edge of the vehicle floor with the door in the open position. Some LRVs have thresholds that project beyond the face of the vehicle so that the clearance between the platform and the carbody may legitimately be in excess of the ADAAG dimension.

- Persons entering a building normally expect a slight step upward, not down, and expect to be stepping down when exiting. Because of this human nature factor, it is important that the vehicle floor never be below the platform. Therefore, the vehicle floor elevation should generally be slightly higher than the station platform elevation so that disembarking patrons have a slight step down.

To properly address ADAAG requirements, designers will consider all dimensional tolerances of the platform/vehicle interface, such as

- Track-to-platform clearances.
- Vehicle-to-track clearances.
- Vehicle dimensional tolerances, new/worn.
- Vehicle load leveling.

The tight horizontal and vertical clearance requirements between the vehicle door threshold and the platform edge impact the construction of track. To maintain these tolerances, some properties have used rigid trackforms to structurally connect the track and the platform. Others seek to only deter ballasted track from lateral movement toward the platform by using extra length crossties butted against the platform foundation wall.

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Chapter 3—Light Rail Transit Track Geometry

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CHAPTER 3—LIGHT RAIL TRANSIT TRACK GEOMETRY

3.1 INTRODUCTION

The most efficient track for operating any railway is straight and flat. Unfortunately, most railway routes are neither straight nor flat. Tangent sections of track need to be connected in a way that steers the train safely and ensures that the passengers are comfortable and the cars and track perform well together. This dual goal is the subject of this chapter.

3.1.1 Design Criteria—General Discussion

The primary goals of geometric criteria for light rail transit are to provide cost-effective, efficient, and comfortable transportation while maintaining adequate factors of safety with respect to overall operations, maintenance, and vehicle stability. In general, design criteria guidelines are developed using accepted engineering practices and the experience of comparable operating rail transit systems.

Light rail transit (LRT) geometry standards and criteria differ from freight or commuter railway standards, such as those described in applicable sections of the American Railway Engineering and Maintenance-of-Way Association's (AREMA's) *Manual for Railway Engineering* (MRE), Chapter 5, in several important aspects. Although the major principles of LRT geometry design are similar or identical to that of freight/commuter railways, the LRT must be able to safely travel through restrictive alignments typical of urban central business districts, including rights-of-way shared with automotive traffic. Light rail vehicles are also typically designed to travel at relatively high operating speeds in suburban and rural settings. AREMA Committee 12 is in the process of adding such information to MRE Chapter 12. However, as of 2011, that process is incomplete.

The LRT alignment corridor is often predetermined by various physical or economic considerations inherent to design within urban areas. One of the most common right-of-way corridors for new LRT construction is an existing or abandoned freight railway line.[1] However, while the desirable operating speed of the LRT line is usually 40 to 55 mph [65 to 90 km/h] or higher, many old rail corridors in densely developed urban areas were originally configured for much slower speeds, often 30 mph [45 km/h] or less.

3.1.2 Design Criteria Development

General guidelines for the development of horizontal alignment criteria should be determined before formulating any specific criteria. This includes knowledge of the vehicle configuration and a general idea of the maximum operating speeds. Design speed is usually defined in terms of what is desirable whenever possible—typically 55 mph [90 km/h]—tempered by a realistic evaluation of what is actually achievable within a given corridor. Physical constraints along various portions of the system, together with other design limitations, may preclude achievement of the desirable speed objective over a significant percentage of the length of the route. Sharp curves in areas of constrained right-of-way are an obvious example. Also, where the LRT operates within a municipal right-of-way, either in or adjacent to surface streets, the maximum operating speed for the track alignment might be limited to the legal speed of the parallel street

traffic even if the track itself is capable of higher speeds. The civil design speed should also be coordinated with the operating speeds used in any train performance simulation program speed-distance profiles as well as with the train control system design.

Where the LRT system includes at-grade segments where light rail vehicles will operate in surface streets in mixed traffic with rubber-tired vehicles, the applicable geometric design criteria for such streets will need to be met in the design of the track alignment.

Where the LRT system includes areas where light rail vehicles will operate in joint usage with railroad freight traffic, the applicable minimum geometric design criteria for each type of rail system needs to be considered. The more restrictive criteria will then govern the design of the track alignment and clearances.

In addition to the recommendations presented in the following articles, it should be noted that combinations of minimum horizontal radius, maximum grade, and maximum unbalanced superelevation are to be avoided in the geometric design.

The geometric guidelines discussed in this chapter consider both the limitations of horizontal, vertical, and transitional track geometry for cost-effective designs and the ride comfort requirements for the LRT passenger.

3.1.3 Minimum and Maximum Criteria Limits

In determining track alignment, several levels of criteria may be considered.[4] Note that an individual criteria limit could be either a minimum or a maximum. In the case of a curve radius, a minimum value would be the controlling limit. In the case of track gradient, there may both a maximum and a minimum—the maximum being controlled by the vehicle's capabilities and the minimum defining the minimum slope necessary to achieve storm water drainage. However, three conditions should be considered: the desirable condition, the acceptable condition, and the absolute condition, each as defined below.

- *Desired Minimum or Maximum*—This criterion is based on an evaluation of maximum passenger comfort, initial construction cost, and maintenance considerations on ballasted, embedded, and direct fixation track. It is used where no physical restrictions or significant construction cost differences are encountered. An optional "preferred" limit may also be indicated to define the most conservative possible future case; i.e., maximum future operating speed for given conditions within the alignment corridor.
- *Acceptable Minimum or Maximum*—This threshold defines a level that, while less than ideal, is considered to be "good enough" to meet the operating objectives without either compromising ride quality or taxing the mechanical limits of the vehicle. The use of acceptable criteria limits typically does not require the designer to produce detailed explanations of why it wasn't possible to do better. Determination of the limits for acceptable criteria is usually project-specific and driven by an interest in maintaining a specific level of service to the maximum degree possible at reasonable cost. As such, the limits of acceptable criteria may be established by qualitative methods rather than a rigorous quantitative analysis.

- *Absolute Minimum or Maximum*—Where physical restrictions prevent the use of both the desired and acceptable criteria, an absolute criterion is often specified. This criterion is determined primarily by the vehicle design, with passenger comfort a secondary consideration. The use of an absolute minimum or maximum criterion should be a last resort. The need for doing so should be thoroughly documented in the project's basis-of-design report and accepted by the project owner.

In addition to the above, lower thresholds of criteria are often stipulated for conditions where ordinary operating speeds are much lower than the desired figures noted above and/or site constraints are extraordinary. These include

- *Main Line Embedded Track*—Where the LRT is operated on embedded track in city streets, with or without shared automotive traffic, there generally are multiple physical site restrictions. Overcoming these requires a special set of geometric criteria that accommodates existing roadway profiles, street intersections, and narrow horizontal alignment corridors that are typical of urban construction and also recognizes the municipal or state design criteria for the roadway surface.
- *Yard and Non-Revenue Track*—These criteria are generally less stringent than main line track, due to the low speeds and low traffic volumes of most non-revenue tracks. The minimum criteria are determined primarily by the vehicle design, with little or no consideration of passenger comfort.

Some yard and non-revenue track criteria may not be valid for frequently used tracks such as when the yard's main entrance leads to and from the revenue service line. For all types of track, the criteria should consider that work train equipment will occasionally use the tracks.

The use of absolute minimum and absolute maximum geometric criteria, particularly for horizontal alignment, has several potential impacts in terms of increased annual maintenance, noise, and vehicle wheel wear, and shorter track component life. The use of any "absolute" criterion should therefore be done only with extreme caution. One or two isolated locations of high track maintenance may be tolerated and included in a programmed maintenance schedule, but extensive use of absolute minimum design criteria can result in revenue service degradation and unacceptable maintenance costs, in both the near term and far term. Designers should therefore attempt to either meet or do better than the "desired" criteria limits whenever it is feasible to do so.

3.2 LRT TRACK HORIZONTAL ALIGNMENT

The horizontal alignment of track consists of a series of tangents joined to circular curves, preferably with spiral transition curves. Track superelevation in curves is used to maximize vehicle operating speeds wherever practicable.

An LRT alignment is often constrained by both physical restrictions and minimum operating performance requirements. This generally results in the effects on the LRT horizontal alignment and track superelevation designs discussed below.

All other things being equal, larger radii are always preferable to tighter turns. In addition to wear and noise, small radius curves limit choices on the vehicle fleet both now and in the future. The

minimum main line horizontal curve radius on most new LRT systems is usually 82 feet [25 meters], a value that is negotiable by virtually every available vehicle. Some modern LRVs and streetcars can negotiate curves as tight as 59 feet [18 meters], and a few can negotiate much smaller radii. Vintage streetcars, including both heritage equipment and modern replicas, can usually negotiate curves as tight as 35 feet [10.7 meters].

Superelevation unbalance (also variously known as "underbalance," "cant deficiency," or simply "unbalance") can range from 3 to 9 inches [75 to 225 mm] depending on vehicle design and passenger comfort tolerance.[3] Vehicle designs that can handle higher superelevation unbalance can operate at higher speeds through a given curve radius and actual superelevation combination. LRT design criteria for maximum superelevation unbalance vary appreciably from as low as 3 inches [75 mm] on some projects to as high as 4 ½ inches [115 mm] on others. The latter value is consistent with a lateral acceleration of 0.1 g, a common, albeit conservative, metric also cited in most design criteria manuals. See Article 3.2.4 for additional discussion on this topic.

LRT spiral transition lengths and superelevation runoff rates are generally shorter than corresponding freight/commuter railway criteria.

The recommended horizontal alignment criteria herein are based on the LRT vehicle design and performance characteristics described in Chapter 2.

The limiting factors associated with alignment design can be classified as shown in **Table 3.2.1**.

3.2.1 Minimum Tangent Length between Curves

The discussion of minimum tangent track length is related to circular curves (see Article 3.2.3). The complete criteria for minimum tangent length will be developed here and referenced from other applicable sections.

The development of this criterion usually considers the requirements of the AREMA *Manual for Railway Engineering*, Chapter 5, which specifies that the minimum length of tangent between curves is equal to the longest car that will traverse the system.[5] This usually translates into a desired minimum criterion of 100 feet [30 meters]. However, that limitation generally addresses operation of freight equipment at low speeds, such as in a classification yard. For passenger operation, ride comfort criteria must be considered. Considering the ability of passengers to adjust for changes of direction, the minimum length of tangent between curves is usually given as

$$L_T = 3V \quad [L_T = 0.57V]$$

where

L_T = minimum tangent length in feet [meters]

V = operating speed in mph [km/h]

This formula is based on vehicle travel of at least 2 seconds on tangent track between two curves. This same criterion also applies to the lengths of circular curves, as indicated below. This criterion has been used for various transit designs in the United States since BART in the

early 1960s.[6] The desired minimum length between curves is thus usually expressed as an approximate car length or in accordance with the formula above, whichever is larger.

Table 3.2.1 Alignment design limiting factors

Alignment Component	Major Limiting Factors
Minimum Length between Curves	• Passenger comfort • Vehicle truck/wheel forces • Vehicle twist
Circular Curves (Minimum Radius)	• Trackwork maintenance • Vehicle truck/wheel forces • Noise and vibration issues
Compound and Reverse Circular Curves	• Passenger comfort • Vehicle frame forces
Spiral Transition Curve Length	• Passenger comfort • Vehicle twist limitations • Track alignment maintenance
Superelevation	• Passenger comfort • Vehicle stability
Vertical Tangent between Vertical Curves	• Passenger comfort
Vertical Curve/Grade (Maximum Rate of Change)	• Passenger comfort • Vehicle frame forces
Special Trackwork	• Passenger comfort • Trackwork maintenance • Noise and vibration issues • Vehicle twist (especially at “jump frogs”)
Station Platforms	• Vehicle clearances • ADAAG platform gap requirements
Joint LRT/Freight RR Usage	• Trackwork maintenance • Railroad alignment criteria • Compatibility of LRT and freight vehicle truck/wheels • Special trackwork components and geometry

Main line absolute minimum tangent length depends on the vehicle and degree of passenger ride quality degradation that can be tolerated. One criterion is the maximum truck center distance plus axle spacing, i.e., the distance from the vehicle’s front axle to the rear axle of its second truck. In other criteria, the truck center distance alone is sometimes used. When spiral curves are used, the difference between these two criteria is not significant.

An additional consideration for ballasted trackwork is the minimum tangent length for mechanized lining equipment, which is commonly based on multiples of 31-foot [10-meter] chords. Very short

curve lengths have been noted to cause significant alignment throw errors by automatic track lining machines during surfacing operations. The 31-foot [10-meter] length can thus be considered an absolute floor on the minimum tangent distance for ballasted main line track in lieu of other criteria.

The preceding discussion is based on reverse curves. For curves in the same direction, it is preferable to have a compound curve, with or without a spiral transition curve, than to have a short length of tangent between the curves. The latter condition, known as a "broken back" curve, does not affect safety or operating speeds, but it does create substandard ride quality. As a guideline, curves in the same direction should preferably have no tangent between curves or, if that is not possible, the same minimum tangent distance as is applicable to reverse curves.

In embedded trackwork on city streets and in other congested areas, it may not be feasible to provide minimum tangent distances between reverse curves. Unless the maximum vehicle coupler angle is exceeded, one practical solution to this problem is to waive the tangent track requirements between curves if operating speeds are below about 20 mph [30 km/h] and no track superelevation is used on either curve.[4] However, the designer must carefully consider unavoidable cross slope that is placed in the street pavement to facilitate drainage and whether light rail vehicle twist limitations might be exceeded. Pavement cross slope can have a direct effect on actual superelevation (Ea) and unbalanced superelevation (Eu) and must be considered when computing minimum spiral lengths. See Article 3.2.9 for additional discussion on this topic.

For yards and in special trackwork, it is very often not practicable to achieve the desired minimum tangent lengths. AREMA *Manual for Railway Engineering*, Chapter 5, provides a series of minimum tangent distances based on long freight car configurations and worst-case coupler angles. It is also noted in the AREMA *Manual for Railway Engineering* that turnouts to parallel sidings can also create unavoidable short tangents between reverse curves. The use of the AREMA table would be conservative for an LRT vehicle, which has much shorter truck centers and axle spacings than a typical freight railroad car. As speeds in yards are restricted by operating rules and superelevation is generally not used, very minimal tangent lengths can be employed between curves. However, because yards typically lack a train control system that would monitor and limit speed, train velocities appreciably higher than those authorized can occur. For this reason alone, compromising on criteria is discouraged.

Existing LRT criteria do not normally address minimum tangent lengths at yard tracks, but leave this issue to the discretion of the trackwork designer and/or the individual transit agency. To permit the use of work trains and similar rail-mounted equipment that are designed around standards for railroad rolling stock, it is prudent to utilize the AREMA minimum tangent distances between reverse curves in yard tracks.

Extremely small radius reverse curves, such as those common for streetcar operations, have an additional consideration. Whenever one light rail vehicle is pushing or towing another, such as commonly occurs around a yard and shop, the angle that the couplers assume to the long axis of both cars must not exceed the vehicles' design limits. A maximum angle of 30 degrees is acceptable, but less would be desired. An angle of 45 degrees to the vehicle should be considered an absolute maximum since, beyond that threshold, the force component tending to push or pull the dead car along the track will be less than the force component that acts to push or pull the vehicle

laterally and hence off the track. One project included an alignment where, during pre-revenue service testing, it was discovered that the tow bar between the streetcar being pushed and the streetcar doing the pushing was at an angle of nearly 90 degrees, at which point all forward motion obviously ceased. The alignment needed to be reconstructed to achieve a smaller angle.

Curves with no intervening tangent are discouraged but can be employed under strict circumstances as described in Article 3.2.7 of this chapter.

Considering the various criteria discussed above for tangents between reverse curves, the following is a summary guideline criteria for light rail transit.

Main Line Desired Minimum

The greater of either

L_T = 200 feet [60 meters] or

L_T = 3V [L_T = 0.57V]

where

L_T = minimum tangent length in feet [meters]

V = maximum operating speed in mph [km/h]

Main Line Acceptable Minimum

The greater of either

L_T = length of LRT vehicle over couplers in feet [meters] or

L_T = 3V [L_T = 0.57V]

where

L_T = minimum tangent length in feet [meters]

V = maximum operating speed in mph [km/h]

Note: So as to not limit future vehicle purchases, the vehicle length is often rounded up for purposes of the equation above. If the actual vehicle is about 90 feet [27 meters] long, the value used in the equation might be 100 feet [30 meters].

Main Line Absolute Minimum

The greater of either

L_T = 31 feet [9.5 meters] or

L_T = (Vehicle Truck Center Distance) + (Axle Spacing)

where the maximum speed is restricted as follows:

$V_{MAX} = L_T / 3$ [$V_{MAX} = L_T / 0.57$]

or

L_T = zero

where the curves meet at a point of reverse spirals, and the spiral lengths and actual superelevation Ea meet the following equation:

$$L_{S1} \times Ea_2 = L_{S2} \times Ea_1$$

where

L_{S1} = length of spiral on the first curve

L_{S2} = length of spiral on the second curve

and maximum vehicle twist criterion is not exceeded. Speed will be limited by the acceptable limits for Eu in the adjoining curves. See Article 3.2.7 for additional discussion of reverse spiraled curves.

Yard and Non-Revenue Secondary Track

The use of main line criteria is preferred in secondary track. When that's not possible, the acceptable minimum tangent lengths would be the smaller of either

L_T = 31 feet [9.5 meters]

or

L_T = zero feet [meters] for R > 950 feet [290 meters]

L_T = 10 feet [3.0 meters] for R > 820 feet [250 meters]

L_T = 20 feet [6.1 meters] for R > 720 feet [220 meters]

L_T = 25 feet [7.6 meters] for R > 640 feet [195 meters]

L_T = 30 feet [9.1 meters] for R > 573 feet [175 meters]

where the specified radius is the smaller of the two curves. Note that the radii thresholds stipulated above are approximations; hence the conversions between U.S. customary and S.I. units are somewhat coarse. Common sense should be exercised in the application of these rules.

Where nothing else will work, the absolute minimum will be

L_T = zero

provided coupler angles are not exceeded, superelevation is zero, and unbalanced superelevation in both curves is 2 inches [50 mm] or less.

3.2.2 Speed Criteria—Vehicle and Passenger

3.2.2.1 Design Speed—General

Desirable LRT operating speeds are in the range of 40 to 55 mph [65 to 90 km/h]. Some LRT projects have used speeds as high as 66 mph [106 km/h]. However, few LRT projects have sufficient tangent track, flat curves, and unrestricted right-of-way for higher speeds to result in meaningful travel time savings. Restricted operating speeds are always possible at discrete points along the alignment corridor, but, for a stadtbahn-type operation, proposed design speeds

below 40 mph [60 km/h] generally create unacceptable constraints on the train control design and proposed operations.

Streetcar/strassenbahn-type LRT operations are generally much slower. It is often presumed that maximum speed in embedded track needs to be restricted, and 35 mph [55km/h] is often cited as a maximum. This is not quite correct. It is not the embedded trackform that limits speed rather than the operating environment surrounding it. Speeds up to the vehicle's maximum can be achieved on embedded track if the guideway is configured appropriately. The reason shared-lane, embedded track is likely to be operated more slowly than track in an exclusive lane is because of traffic conditions, adjacent parking lanes, pedestrian crosswalks, and other community-related issues. Some legacy streetcar lines that operated in shared lanes along wide-open streets and boulevards routinely operated at the vehicles' balancing speed—sometimes as fast as 40 to 50 mph [65 to 80 km/h].

There is a requirement in the *Manual of Uniform Traffic Control Devices* (MUTCD) that requires LRT crossings to be equipped with flashing lights if trains are running faster than 35 mph [55 km/h]. Technically, that rule has no effect on what happens between intersections, although transit agencies may elect to limit speed in such areas merely to avoid cycles of acceleration and deceleration when passing through a multiple crossing zone. Furthermore, if the LRT is in a mixed traffic lane, flashing light signals and gates would be completely impractical at each intersecting street regardless of speed. As of 2010, TCRP Project A-32 is investigating the MUTCD requirement for railroad-style warning systems at LRT crossings. Users of this Handbook should consult the TCRP program and the current edition of the MUTCD for the latest information. See Chapter 10 for additional discussion on this topic.

3.2.2.2 Design Speed in Curves

The speed criteria for curved track is determined by carefully estimating passenger comfort and preventing undue forces on the trackwork, vehicle trucks/wheels, and vehicle frames. Vehicle stability on curved track is also an important consideration in the determination of LRT speed criteria.

Curved track that cannot be used at the same speed as the adjoining tangent track slows down the operation by increasing the overall running time between terminals. This wastes kinetic energy in the form of the momentum the vehicle had prior to slowing down and requires the consumption of additional energy to speed back up. It takes more than 0.62 mile [1 kilometer] for a rail vehicle to decelerate from 70 mph [110 km/h] to 55 mph [90 km/h], run through a 1000 foot [300-meter] long circular curve, and accelerate back up to 70 mph [110 km/h]. The same curve designed for a reduction down to 45 mph [70 km/h] reduces the speed over a length of about 0.75 mile [1.2 kilometers]. The actual increase in running time is relatively small, but cumulative run time losses at successive curves can significantly increase the overall travel time from terminal to terminal.

Repetitive slowing down and speeding back up often annoys passengers (particularly standees) by subjecting them to a jerky ride. This unpleasant experience could have an effect on individuals' subsequent personal decisions as to whether or not to ride transit. Such a ride also causes additional wear and tear on both the vehicle and the track.

Therefore, it is generally desirable to eliminate as many speed restrictions as possible and to maximize the design speed of all curves that must unavoidably be designed with speed restrictions. This can be achieved in three ways:

- Using curve radii that are as broad as possible. This is the preferred method, but not always practical within the constraints of available right-of-way.
- Maximizing the speed on the curves by introducing actual superelevation (Ea) in the track and maximizing the value of unbalanced superelevation (Eu) used.
- Combinations of the above.

See Article 3.2.6 for additional discussion on determination of appropriate speeds in curved track.

3.2.3 Circular Curves

Intersections of horizontal alignment tangents are connected by circular curves. The curves may be simple curves or spiraled curves, depending on the curve location, curve radius, and required superelevation. In very nearly all cases, spiraled curves are preferable so as to improve ride quality and minimize impacts to rolling stock.

3.2.3.1 Curve Radius and Degree of Curve

LRT alignment geometry differs from freight railroad design standards such as AREMA in that the arc definition is used to define circular curves. Also, curves for LRT designs are generally defined and specified by their radius rather than degree of curvature. This becomes an important distinction when designing in metric units, as degree of curve is defined entirely in traditional U.S. units and has no direct equivalent in metric units.

Railroads have traditionally employed the chord definition of degree of curvature for calculating curves. The reasons for this practice date back to the surveying equipment and centerline stakeout methods that were employed during the mid-19th century. Railroads have persisted in requiring the chord definition for new railroad design despite radical advances in surveying methods. However, rail transit in general and light rail in particular use curve radii that are so sharp as to make degree of curvature impractical for ordinary use. For this reason, arc definition with lengths computed along the centerline of the curve is recommended for LRT design.

Modern computer-aided design and drafting (CADD) alignment computation software can easily compute curvature in either arc or chord definition. Any curves that have been computed using the chord definition should be clearly labeled as such on the plan and profile drawings.

In the case of any project to be designed using S.I. units of measurement but utilizing an existing right-of-way that is based on traditional U.S. units, particularly the degree of curvature, it is most efficient to determine the radius in traditional U.S. units, and then to convert to metric.

As a guideline for LRT design, curves should be specified by their radius. Degree of curvature, when needed for calculation purposes, should be defined by the arc definition of curvature as determined by the following formula:

$$Da = 5729.58 / R$$

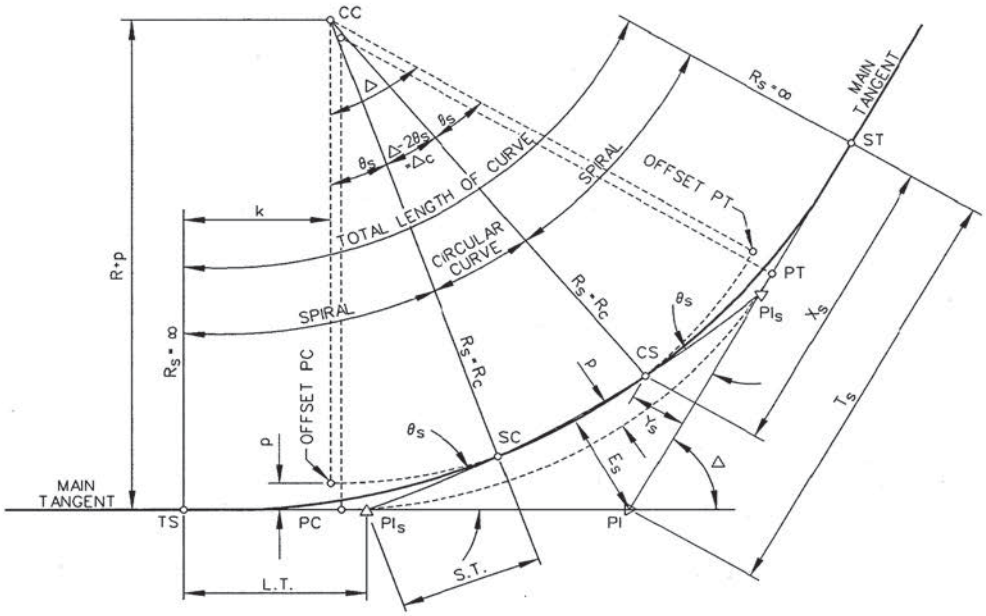
where Da is the degree of curve using the arc definition and R is the radius in feet. There is no equivalent formula using S.I. units since degree of curvature is not used in metric design.

3.2.3.2 Minimum Curve Radii

Circular curves for LRT design are, as noted above, defined by curve radius and arc of curve length. The geometric properties of the circular curve are summarized in **Figure 3.2.1**.

The straighter the route, other factors being equal, the less maintenance it will require. For this reason, the designer should seek alignments that minimize curves, especially very sharp curves. The minimum curve radius is determined by the physical characteristics of the vehicle. For most modern LRV designs, whether high- or low-floor, the most common absolute minimum radius is 82 feet [25 meters]. Some vehicles can negotiate curves with radii of 59 feet [18 meters]. A very few vehicles can negotiate even smaller curves. Light rail vehicles in Boston and San Francisco go around radii of 42 feet [12.8 meters], and legacy streetcars in hundreds of cities and towns throughout the United States routinely traversed curves with radii of 35 feet [10.7 meters]. However, while extremely tight curves are possible, they limit carbuilders' options and hence the universe of candidate LRVs that could be used on a system. The use of curves tighter than 82 feet [25 meters] is therefore strongly discouraged. Refer to Chapter 2 for additional information on vehicle limitations. Refer to Chapter 12 for additional discussion on use of small radius curves in urban areas.

On-track maintenance-of-way (M/W) equipment must also be considered in the selection of minimum horizontal curve criteria. Depending on the maintenance plan for the system, this could include a wide variety of hy-rail trucks, tampers, ballast regulators, ballast cars, catenary maintenance vehicles, and even small locomotives. It is highly desirable for the alignment to allow such equipment to operate from the maintenance depot to any point on the LRT system where they might be used. This affects track geometry, clearance, and trackwork issues. For example, a segment of sharply curved, embedded track located at a midpoint of the system may make it impossible for M/W equipment to access one end of the route from a yard and shop on the opposite end of the line. This could have a distinct impact on the equipment requirements for supporting the LRT maintenance-of-way plan. Curves that cannot be negotiated by the M/W fleet are therefore optimally confined to tracks where on-track access is not essential, such as terminal loops and yard turnaround tracks that can be serviced using off-track roadways.[12]



NOTATIONS

CC = CENTER OF CIRCULAR CURVE

CS = POINT OF CHANGE FROM CIRCULAR CURVE TO SPIRAL

D_c = DEGREE OF CIRCULAR CURVE, ARC DEFINITION

E_s = TOTAL EXTERNAL DISTANCE OF A SPIRALIZED CURVE

k = TANGENT DISTANCE FROM TS OR ST TO PC OR PT OF THE SHIFTED CIRCULAR CURVE

L_c = TOTAL LENGTH OF CIRCULAR CURVE ARC

L_s = TOTAL LENGTH OF SPIRAL

L.T. = LONG TANGENT OF SPIRAL

p = OFFSET FROM THE MAIN TANGENT TO THE PC OR PT OF THE SHIFTED CIRCULAR CURVE

PC = POINT OF CHANGE FROM TANGENT TO CIRCULAR CURVE

PI = POINT OF INTERSECTION OF MAIN TANGENTS

PI_s = POINT OF INTERSECTION OF MAIN TANGENT WITH TANGENT THROUGH SC OR CS POINT

PT = POINT OF CHANGE FROM CIRCULAR CURVE TO TANGENT

R = RADIUS OF CIRCULAR CURVE

SC = POINT OF CHANGE FROM SPIRAL TO CIRCULAR CURVE

ST = POINT OF CHANGE FROM SPIRAL TO TANGENT

S.T. = SHORT TANGENT OF SPIRAL

T_s = TOTAL TANGENT DISTANCE FROM TS OR ST TO PI

TS = POINT OF CHANGE FROM TANGENT TO SPIRAL

X_s = TANGENT DISTANCE FROM TS TO SC OR ST TO CS

Y_s = TANGENT OFFSET AT SC OR CS

Δ = TOTAL CENTRAL ANGLE OF SPIRAL AND CIRCULAR CURVES

Δ_c = CENTRAL ANGLE OF THE CIRCULAR CURVE

θ_s = CENTRAL ANGLE OF SPIRAL

CURVE FORMULAS

$$D_c = \frac{5729.578}{R}$$

$$T_s = (R+p)\tan\frac{\Delta}{2} + k$$

$$E_s = (R+p)\left(\frac{1}{\cos\frac{\Delta}{2}} - 1\right) + p$$

$$L_c = \frac{\Delta_c}{D_c} \times 100 = \frac{\Delta - 2\theta_s}{D_c} \times 100$$

SPIRAL FORMULAS
θ_s IN RADIANS

$$X_s = L_s\left(1 - \frac{\theta_s^2}{10} + \frac{\theta_s^4}{216} - \frac{\theta_s^6}{9360} \cdots\right)$$

$$Y_s = L_s\left(\frac{\theta_s}{3} - \frac{\theta_s^3}{42} + \frac{\theta_s^5}{1320} - \frac{\theta_s^7}{75600} \cdots\right)$$

$$k = L_s\left(\frac{1}{2} - \frac{\theta_s^2}{60} + \frac{\theta_s^4}{2160} - \frac{\theta_s^6}{131040} \cdots\right)$$

$$p = L_s\left(\frac{\theta_s}{12} - \frac{\theta_s^3}{336} + \frac{\theta_s^5}{15840} \cdots\right)$$

$$L_s = 2R\theta_s$$

$$\theta_s = \frac{1}{2}\frac{L_s}{R}$$

$$L.T. = X_s - \frac{Y_s}{\tan\theta_s}$$

$$S.T. = \frac{Y_s}{\sin\theta_s}$$

Figure 3.2.1 Horizontal curve and spiral nomenclature

One frequently employed criterion for the desired minimum curve radius is the threshold limit for employing restraining rail, as determined from Chapter 4. In many cases, this is around 500 feet [150 meters]. Other possible thresholds for desirable minimum radius are either the limit selected for employing premium rail versus standard strength rail or the limit between the use of plain continuously welded rail (CWR) versus shop-curved rail. Sometimes a slight increase in radius will eliminate the need to utilize a more expensive trackform. Carrying that thought beyond trackwork costs, it should also be noted that sharply curved tunnels and aerial structures can have significantly higher construction costs than similar structures on tangent track or flat curves.

In view of the design considerations indicated above, guideline criteria for modern LRV equipment are as follows for minimum curve radii.

Main Line Track

At-Grade Acceptable Minimum. Greater of

- 500 feet [150 meters] or
- Threshold radius for employment of more expensive trackforms.

Tunnels and Aerial Structures Acceptable Minimum. Greater of

- 500 feet [150 meters] or
- Other value as suggested by the project's structural designers.

Ballasted At-Grade Track, Absolute Minimum.

- 300 feet [90 meters]

Embedded Track or Direct Fixation Track, Absolute Minimum. Lesser of

- 82 feet [25 meters] or
- Other value as permitted by the vehicle design.

Yard and Non-Revenue Secondary Track

Acceptable Minimum. Lesser of

- 100 feet [30 meters] or
- Other value as required by the vehicle design.

Absolute Minimum. Lesser of

- 82 feet [25 meters] or
- Other value as required by the vehicle design.

3.2.3.3 Minimum Curve Length

The minimum circular curve length is dictated by ride comfort and is, hence, unlike minimum tangent length, not related to vehicle physical characteristics. The acceptable minimum circular curve length is generally determined by the following formula:

$$L = 3V \qquad [L = 0.57V]$$

where L = minimum length of curve in feet [meters]
V = design speed through the curve in mph [km/h]

For spiraled circular curves in areas of restricted geometry, the length of the circular curve added to the sum of one-half the length of both spirals is an acceptable method of determining compliance with the above criteria. The absolute minimum length of a superelevated circular curve should be approximately 10 to 15 feet [3 to 5 meters] longer than the truck center distance on the light rail vehicle so that the vehicle is not simultaneously twisting through two superelevation transitions. In such cases, a speed restriction should be imposed based on the formula above.

Curves that include no actual circular curve segment (e.g., double-spiraled curves) should be permitted only in areas of extremely restricted geometry (such as embedded track in an urban area), provided no actual superelevation (E_a) is used. This type of alignment is potentially difficult to maintain for ballasted track.

The design speed for a given horizontal curve should be based on its radius, length of spiral transition, and the actual and unbalanced superelevation through the curve as described in the following sections.

3.2.4 Curvature, Speed, and Superelevation—Theory and Basis of Criteria

This article summarizes the basis of design for determination of speed and superelevation in curved track. This material is based on information provided by Nelson,[7] but has been condensed and modified as necessary for specific application to current LRT designs and to include the use of metric units.

3.2.4.1 Superelevation Theory

The design speed at which a light rail vehicle can negotiate a curve is increased proportionally by increasing the elevation of the outside rail of the track, thus creating a banking effect called superelevation.

When rounding a curve, a vehicle and the passengers within it are subjected to lateral acceleration acting radially outward. The forces acting on the vehicle are illustrated in **Figure 3.2.2**.

Ride comfort criteria, including making certain that any standing passengers on the rail vehicle do not fall, requires limiting train speed so that lateral acceleration does not exceed certain thresholds. This is traditionally expressed in terms of a fraction of the acceleration of gravity. The traditional value used was one-tenth the acceleration of gravity, or 0.1 g. That value, which was empirically derived from studies dating back to the beginning of the 20th century, is a conservative value and good for ordinary applications. More recent research has indicated that higher values can be tolerated. Lateral acceleration as high as 0.15 g has been successfully used on some high-speed railways and can be used for rail transit under the following circumstances:

- Spirals of appropriate length are provided to limit jerk.
- The trackform is rigid, such as either direct fixation track or embedded track, so that deterioration of track geometry is nearly impossible. Use of high values of lateral acceleration in ballasted track will require extraordinary maintenance attention to track surfacing and crosslevel so as to avoid misalignments that result in values of lateral acceleration higher than 0.15 g.

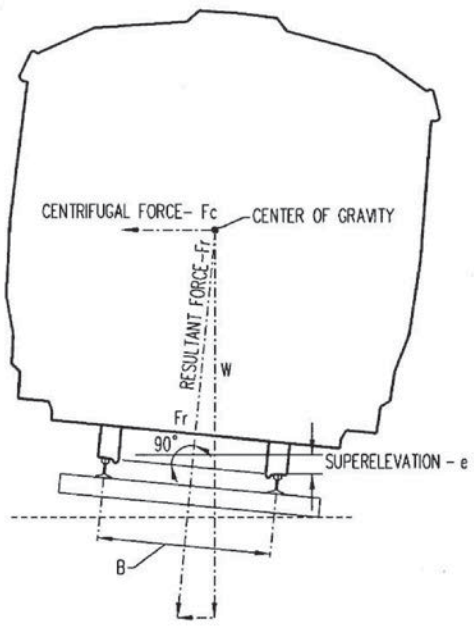


Figure 3.2.2 LRT vehicle on superelevated track

To counteract the effect of the lateral acceleration and the resulting centrifugal force (F_c), the outside rail of a curve is raised by a distance above the inside rail 'e'. A state of equilibrium is reached in which both wheels exert equal force on the rails, i.e., where 'e' is sufficient to bring the resultant force (F_r) to right angles with the plane of the top of the rails.

The AREMA *Manual for Railway Engineering*, Chapter 5, gives the following equation to determine the distance that the outside rail must be raised to reach a state of equilibrium, where both wheels bear equally on the rails:

$$e = \frac{B V^2}{g r}$$

where

- e = equilibrium superelevation in feet or meters. (Note: not inches or mm in this formula)
- B = bearing distance of track in feet or meters. This value is equal to the track gauge plus the distance to the center of the railheads. The absolute value will therefore be different for standard gauge, broad gauge, and narrow gauge tracks.
- V = velocity in feet [meters] per second. (Note: not mph or km/h in this formula).
- g = acceleration due to gravity in feet per second per second, or feet/sec^2 [meters per second per second, or meters/sec^2].
- r = radius in feet [meters].

To convert these units to common usage:

- 'e' in feet or meters is usually expressed as either 'E' or 'Eq' (preferred) in either inches or millimeters.

- 'B' is usually considered to be 60 inches [1524 millimeters] on standard gauge track; however, the 60-inch value is actually a fairly crude approximation. The actual value, assuming 115RE rail, would be 59 ¼ inches [1505 mm]. Hence, a valid conversion to S.I. units (i.e., one consistent with the tolerances implied by the rounding of 59 ¼ inches up to 60 inches) would be 152 cm (expressed as 1520 mm in the calculations below).

- Vehicle velocity 'V,' expressed in feet per second [meters per second] is changed to 'V' in mph [km/h].

- The acceleration of gravity 'g' is equal to 32.2 feet/sec^2 [9.81 meters/sec^2].

When working in traditional U.S. units, the curve radius 'r' can be replaced with 5729.58/D, where 'D' is equal to the arc definition degree of curvature expressed as a decimal of whole degrees. However, there is no S.I. equivalent for degree of curve. Moreover, since it is extremely rare that an LRT track curve will have a radius exactly equal to some convenient even number degree of curve, it is recommended that these calculations be based on the radius in feet and decimals of a foot.

The AREMA formula hence can be expressed as follows:

$$E = \frac{59.25\ V^2}{(32.2)\ R\ \left(\frac{3,600}{5,280}\right)^2} = \frac{3.96\,V^2}{R} \left[E = \frac{1,520\ V^2}{(9.81)\ \ R\ \ \left(\frac{3,600}{1,000}\right)^2} = \frac{11.96\ V^2}{R} \right]$$

The traditional U.S. units version of the equation above is sometimes seen as $E = 4.01\ V^2/ R$. That occurs when the designer used the rough 60-inch value for the bearing distance as opposed to the somewhat more accurate 59 ¼ inches. In truth, given the rounding of the actual value of g used in the development of the equations, the fact that the bearing distance will vary depending on both the rail section used and the wear on both rails and wheels, plus the rational construction and maintenance tolerances for track gauge, both 3.96 and 4.01 are unnecessarily precise. The same pragmatism can be applied to the S.I. version of the equation. Therefore, the simplified equations:

$$Eq = 4.0\ V^2/ R \text{ (U.S. traditional units)}$$

and

$$Eq = 12.0\ V^2/ R \text{ (S.I. units)}$$

are actually sufficiently accurate for ordinary purposes at the speeds to be encountered in LRT design.

The formulae above compute the amount of superelevation necessary for equilibrium. So as to clearly distinguish that figure from other values of superelevation discussed below, the shorthand designation "Eq" is recommended when discussing superelevation needed for equilibrium.

Experience has shown that safety and comfort can be optimized if vehicle speed and curvature are coordinated such that Eq falls in the range of 3 to 4 ½ inches [75 to 115 mm]. This is an extremely conservative goal and can provide a very gentle ride. However, it is rarely practical to achieve without substantial civil works that are typically well outside the budget for most light rail transit projects. Accordingly, higher values of Eq are typically necessary so as to avoid negative impacts on the LRT system running times from terminal to terminal.

3.2.4.2 Actual Superelevation

The actual value of superelevation installed in the track is typically somewhat less than required for equilibrium. This "actual superelevation" is commonly abbreviated as "Ea." Most railway route design texts recommend an absolute limit of 8 inches [200 mm] of actual superelevation for passenger operations unless slow-moving or freight traffic is mixed with passenger traffic. Values of Ea that large are very seldom used, in part because the passengers on any train that might stop on such a curve would be extremely uncomfortable. Therefore, LRT superelevation is generally limited to 6 inches [150 mm] or less.

All railroads administered by the Federal Railroad Administration (FRA) are limited to no more than 6 inches [150 mm] of Ea, primarily because the FRA mandates that all tracks that are a part of the nation's general railroad system must be capable of handling mixed traffic. Track that is not part of the general railroad system or that is used exclusively for rapid transit service in a metropolitan or suburban area, generally does not fall within the jurisdiction of the FRA. This includes the vast majority of LRT systems. Even in the case of LRT lines that share some track with a freight railroad operation, the FRA might not choose to exercise any authority over LRT tracks that are not used by the freight operator.

In view of the foregoing, railways that are not subject to oversight by the FRA may, when appropriate, use up to 8 inches [200 mm] of actual superelevation on curved track. This has been applied to at least two North American metro rail transit systems. However, it is far more common on LRT systems to limit maximum actual superelevation to 6 inches [150 millimeters], as it becomes more difficult to consistently maintain ride comfort levels at higher actual superelevation, particularly in cases where running speeds may vary.

3.2.4.3 Superelevation Unbalance

The equations in Article 3.2.4.1 above are expressed in terms of a single speed at which the rail vehicle is at equilibrium with the resultant vector, F_r, aimed directly at the centerline of track. However, for a variety of reasons, rail vehicles often run at different speeds on the same segment of track and hence would require some different value of track superelevation for each of those speeds. This is obviously impossible; however, it is perfectly acceptable, within limits, to operate at speeds either greater than or less than the equilibrium speed. When the operating speed is greater than the equilibrium speed, the variance is termed superelevation underbalance. This is sometimes contracted to simply "unbalance." The term "cant deficiency" is also sometimes seen, "cant" being a British vernacular for superelevation. Underbalance is commonly abbreviated as Eu. Operation at speeds less than the equilibrium condition results in "overbalance," which can be considered as "negative" Eu.

Limited superelevation unbalance is intentionally incorporated into most curve design speed calculations to avoid the negative effects of occasional operation at speeds less than equilibrium speed. For rail transit, the principal issue is passenger discomfort; negative Eu is not tolerated well by passengers, who sometimes have the perception that they are falling out of their seats. In freight operations, negative Eu can result in excessive loading of the low (inside) rail of the curve leading to a variety of metallurgical defects. This is generally not an issue with LRT since transit axle loadings are far smaller than those of freight cars.

The development of high-speed intercity passenger rail operations using rolling stock with sophisticated suspension systems has led to extensive research in the field of superelevation and allowable amounts of unbalance. As noted above, high-speed rail operations typically allow higher values of lateral acceleration and hence higher values of Eu.

Ignoring vehicle roll (see Article 3.2.4.4), 0.1 g of lateral acceleration equates to almost exactly 6 inches [150 mm] of unbalance on standard gauge track. Per AREMA, vehicles with stabilized suspensions have vehicle roll (to the outside of the curve) equivalent to about 1 ½ inches [38 mm] of unbalance. Subtracting 1 ½ inches from 6 inches leaves 4 ½ inches [114 mm] for Eu. Hence, any criterion that restricts Eu to be less than 4 ½ inches is actually restricting lateral acceleration to something less than 0.1 g. Nevertheless, maximum allowable superelevation unbalance varies among transit agencies. For instance, a now-obsolete criterion for one large legacy heavy rail transit operator allowed only 1 inch [25 mm] of Eu, while newer systems, beginning with PATCO (the Lindenwold High-Speed Line, which opened in 1968), usually allowed 4 ½ inches [115 mm]. That larger value is consistent with a lateral acceleration of 0.1 g while the obsolete value is equivalent to less than 0.02 g. Generally, it is recognized that 3 to 4 ½ inches [75 to 115 mm] of Eu is acceptable for LRT operations, depending upon the vehicle design.

3.2.4.4 Vehicle Roll

In a curve with no actual superelevation, Ea, all of the lateral acceleration effectively becomes unbalance. Speed then becomes limited by the value selected for lateral acceleration. If the value of lateral acceleration is the customary 0.1 g, the unbalance on standard gauge track works out to 6 inches [150 mm]. At 0.15 g, the unbalance would be 9 inches [230 mm]. However, those values are not actually Eu. To determine Eu, one must first subtract a factor for vehicle roll.

All types of rail vehicles have suspension systems that allow the car or locomotive to react to variations in the track surface and to dampen impacts. A consequence of these suspension systems is that when the vehicle is passing through a curve, it will roll about a rotation point or points within its suspension system. The vehicle will roll toward the outside of the curve until it reaches a point where either the springs in the suspension system counteract the rotating force or the rotation reaches a mechanical stop in the vehicle's trucks.

The AREMA *Manual for Railway Engineering* (2008) Chapter 5, Article 3.3.1, explains:

> *Equipment designed with large center bearings, roll stabilizers and outboard swing hangers can negotiate curves comfortably at greater than 3 inches [75 mm] of unbalanced superelevation because there is less body roll....Lean tests may be made on tangent track by running one side of the car onto oak shims, using winches to move the car on and off the shims. Cars should be elevated to three*

heights: usually 2 inches, 4 inches, and 6 inches [50 mm, 100 mm, and 150 mm]. If the roll angle is less than 1°-30', experiments indicate that cars can negotiate curves comfortably at 4.5 inches [115 millimeters] of unbalanced elevation.

Because the carbody roll in a moving vehicle is toward the outside of the curve, it has the effect of being "negative superelevation" and thus cancels out some portion of either the actual superelevation or unbalanced superelevation of the curve. The 1°30' roll value noted by AREMA, applied over the width of standard gauge track, is effectively equal to 1 ½ inches [about 38 mm] of additional unbalance. So, if the maximum acceptable unbalance for the system based on a lateral acceleration of 0.1 g is 6 inches [152 mm], the value of Eu actually available to the track designer is only 4 ½ inches [about 114 mm]. The difference—call it "superelevation roll," or "Er"—has effectively been appropriated by the vehicle's suspension system.

Naturally, the actual value of Er on any given curve will vary. Depending on the design of the rail vehicle, its maintenance condition, and its instantaneous speed, the actual value of Er could be less than or perhaps even greater than AREMA's figure of 1 ½ inches [38 mm]. Those factors are outside of the track designer's control. There is also a lack of firm data on the roll factor (Er) of various types of light rail vehicles/streetcars. Notably, the possible carbody roll, as indicated by the dynamic envelope for a typical light rail vehicle (see Chapter 2, Figure 2.3.2), is generally much larger than the AREMA figure. This is an area that requires further investigation. In the absence of specific information for the proposed light rail vehicle, the AREMA guidance can be used. However, the track designer should verify with the project's vehicle designers and carbuilders what the maximum carbody roll is for the design vehicle(s). Notably, the AREMA static lean test procedure quoted above is not commonly included in vehicle procurement specifications.

If the vehicle fleet includes any heritage, antique, or replica streetcar equipment, the suspension systems and hence the body roll angle may be appreciably different from that of newer rolling stock. If so, it may be necessary to impose speed restrictions on heritage equipment so as to keep the lateral acceleration at or below the selected value.

Therefore, equilibrium superelevation can be expressed as

$$Eq = Ea + Er + Eu = 4\ V^2 / R$$

$$[Ea = Ea + Er + Eu = 12\ V^2 / R]$$

and the actual superelevation for maximum comfortable speed (Ec) may be expressed as

$$Ec = Eq – Er = Ea + Eu$$

The value of Er, once it has been deducted from the maximum allowable value of superelevation unbalance, is not used in any subsequent calculations. Thus, if an LRT vehicle is of modern design, it is appropriate to use up to 4 ½ inches [114 mm] of Eu as a parameter in the design of track curves. The formulae from Article 3.2.4.1 may therefore be restated as

$$Eq = Ea + Eu = 4.0\ V^2 / R \text{ (U.S. traditional units)}$$

and

$$Eq = Ea + Eu = 12.0\ V^2 / R \text{ (S.I. units)}$$

3.2.4.5 Ratio of Ea to Eu

How to balance Ea and Eu is largely a qualitative decision, and several strategies are employed by different transit agencies:

- No (or minimal) superelevation unbalance is applied until actual superelevation (Ea) reaches the maximum allowable level. Actual superelevation is thus equal to the equilibrium superelevation for most curves. Under ideal conditions, where all vehicles operate at the same speed and do not stop (or slow down) on curves, this strategy creates the least amount of passenger and vehicle lateral acceleration for a given transition curve length. Under less-than-ideal operating conditions, however, the minimum superelevation unbalance strategy produces unfavorable ride comfort conditions.
- No unbalanced superelevation (Eu) until Ea has reached some figure. This recognizes that carbody roll (Er) in response to lateral acceleration is one of the first results of vehicle entry into a curve. By introducing Ea immediately, some of the jerk experienced by the passengers is mitigated, providing for a smoother ride.
- Maximum superelevation unbalance is applied before any actual superelevation is considered. This option is often used by freight and suburban commuter railroads. Where a wide variety of operating speeds is anticipated on the curved track, particularly on joint LRT-freight trackage, this strategy is usually the least disruptive to passenger comfort.
- No actual superelevation Ea until Eu has reached some figure. This simplifies track construction (but not necessarily track maintenance) by eliminating superelevation on large radius curves. This approach is generally not recommended; however, it may become necessary in specific circumstances. For example, when constructing embedded track in a public street, it may not be possible to have any actual superelevation without causing problems with pavement contours and drainage. In such cases, most or all of the value of Eq would be taken up by Er and Eu.
- Actual superelevation (Ea) and superelevation unbalance (Eu) are applied equally or in some proportion. Because a certain amount of superelevation unbalance, applied gradually, is generally considered to be easily tolerated by both vehicle and passenger and tolerable superelevation unbalance increases with speed, this strategy is preferred for general usage.

Other combinations might be considered. For example, it might be considered desirable to ordinarily limit Eu to some fairly low threshold value and blend Ea and Eu up until Ea reaches the maximum. Thereafter, Eu only would be increased until it reached its maximum.

As a practical matter for construction, curves with a large radius in comparison to the desired operating speed should not be superelevated. This can be accomplished by not applying actual superelevation (Ea) until the calculated total equilibrium superelevation (Eq) is over a minimum value, usually ½ to 1 inch [12 to 25 millimeters]. However, despite the lack of Ea, such curves usually still need a spiral so as to counteract the lateral acceleration effects of Eu and Er.

LRT systems are typically operated under the manual control of the vehicle operator, subject to both the commands of the signal systems and printed operating rules. This is distinctly different from modern metro rail systems, where automatic train operation results in the exact same train speeds at any given location a very high percentage of the time. By contrast, LRT train speeds on any given curve can vary over a relatively wide margin from virtually stopped up to the maximum speed permitted by the train control system. Operation at an optimal design speed actually occurs only a fraction of the time. It therefore becomes important to select an appropriate balance between Ea and Eu. If Ea is too high, the passengers on board slow or stopped trains could be uncomfortable. If Eu is too high, passengers will be subjected to a rougher ride than necessary. So as to optimize ride comfort, the normal practice is to introduce Eu and Ea nearly simultaneously. The following example (using traditional U.S. units) illustrates the process given the following design criteria policy decisions:

- Maximum Ea = 6 inches.
- Maximum Eu = 4 ½ inches.
- No Eu until Ea has reached ½ inch.
- Eu and Ea increased linearly once Eu is initiated.

Plotting those parameters, as shown in **Figure 3.2.3**, sets the slope and y-axis intercept of a line defining Eu in terms of Ea.

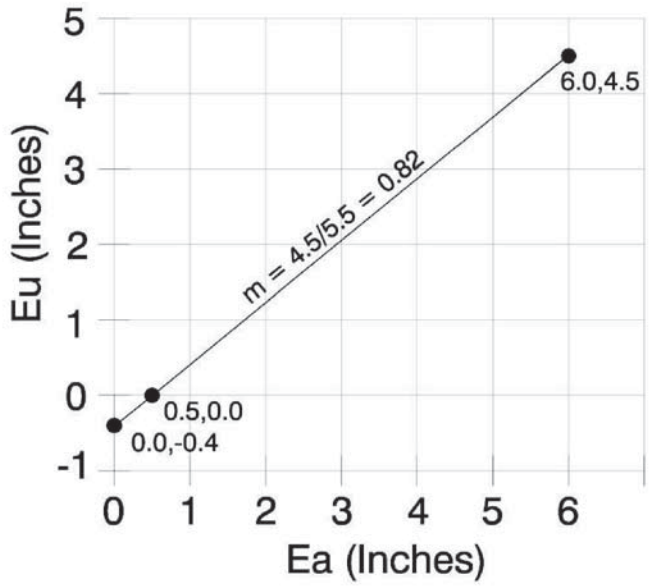


Figure 3.2.3 Example of ratio of Eu to Ea

Mathematizing this line in the classic y = mx + b equation format results in

$$Eu = 0.82\ Ea - 0.4$$

Substituting into the modified AREMA equation developed in Article 3.2.4.1 above:

$$Ea + (0.82\ Ea - 0.4) = 3.96\ V^2 / R$$

and solving for Ea results in

$$Ea = 2.18\ (V^2 / R) - 0.22$$

Subtracting that calculated Ea from Eq then gives the value of Eu.

Naturally, different assumptions concerning maximum values of both Ea and Eu and when Eu should be introduced would result in an appreciably different formula. As an example, one U.S. metro rail transit project very conservatively limited Eu to an maximum of 2 ½ inches [64 mm], held Ea to no more than 6 inches [152 mm], and introduced Eu only after Ea equaled 1 inch [25 mm]. Their version of the previous equation (in traditional U.S. units) therefore became

$$Ea = 2.64\ (V^2 / R) - 0.66$$

Use of equations such as the examples above will result in the gradual introduction of both actual and unbalanced superelevation and avoid unnecessarily high values of lateral acceleration and jerk to both the light rail vehicles and their passengers.

As a practical matter for construction, calculated values for actual superelevation should be rounded up to the next ¼ inch when working in traditional U.S. units. Use 5 mm as the working increment for Ea when using S.I. units. The difference between Eq and that rounded value of Ea becomes the actual Eu at the design speed.

For a total superelevation (Ea + Eu) of 1 inch [25 millimeters] or less, actual superelevation (Ea) is not usually applied. In specific cases where physical constraints limit the amount of actual superelevation (Ea) that can be introduced, a maximum of 1 ½ inch [40 mm] of superelevation unbalance (Eu) is often permitted before applying any actual superelevation (Ea).

On curves where speed is likely to vary, such as on the approaches to passenger stations, the actual superelevation (Ea) is usually set so that trains will have a positive value of superelevation unbalance (Eu). This is because large values of negative Eu (i.e., Ea is greater than Eq) are not tolerated well by passengers. For this reason, consideration should be given to the difference in speed between the front and rear of the train as they pass the cardinal points along the curve.

As noted above, differing circumstances at different locations on the same rail transit project may require different ratios and formulae for balancing Ea and Eu. However, along any given route segment it is desirable to keep them as consistent as is reasonably possible. Individual curves that have a much higher proportion of Eu than other nearby curves could catch passengers unaware and cause incidents. High values of superelevation unbalance increase track/vehicle forces and hence maintenance of both. Conversely, operations closer to balance speed result in a more comfortable ride and less impact on the vehicle and track. Therefore, given consistent speeds and circumstances it is preferable to maximize actual superelevation and minimize

superelevation unbalance to reduce the effects of centrifugal force upon the passengers, vehicles, track structures, and roadbed.

3.2.5 Spiral Transition Curves

When an LRT vehicle operating on straight (tangent) track reaches a circular path, the vehicle axles must be set at a new angle, depending upon the radius of the curve. This movement is not done instantly but over a measurable time interval, thus creating the need for a transitional or easement curve, the length of which equals speed multiplied by time. Superelevated circular curves virtually always require such easement curves so as to control the acceleration and resulting forces exerted upon the track, the passengers, and the vehicle. These easement curves are usually spirals with the radius decreasing from infinity, where they meet the adjoining tangent track, down to the radius of the circular curve being entered. A similar (and usually symmetrical) transition is provided at the exit end of the curve. Spiral curves also provide the ramp for introducing superelevation into the outside rail of the curve. Spirals are also used as transitions between compound circular curves, as discussed in Article 3.2.8.

3.2.5.1 Spiral Application Criteria

Spirals should be used on all main line track horizontal curves with radii less than 10,000 feet [3,000 meters], wherever practicable. For operation at speeds likely to be encountered in LRT design, spirals can be omitted if the calculated length of spiral (Ls) is less than 0.01R, where R is the radius of the curve. (The formula is the same using either feet or meters for both Ls and R.)

A spiral is preferred, but not required, for yard and secondary tracks where design speeds are less than 10 mph [16 km/h]. Curves on yard lead and secondary tracks that have greater design speeds should have spiral transition curves and superelevation.

3.2.5.2 Spirals and Superelevation

Actual superelevation (Ea) should normally be attained and removed linearly throughout the full length of the spiral transition curve by raising the outside rail while maintaining the inside rail at the profile grade. One exception to this customary method of achieving superelevation is sometimes employed for direct fixation tracks in circular tunnels, such as might be created by a tunnel-boring machine, where superelevation is achieved by rotating the track section about the tracks profile grade line. This usually minimizes the overall size of the bored tunnel required through the curve. Since the tunnel diameter, as created by the boring machine, will be the same in both curved and tangent track, a substantial amount of tunnel excavation can be avoided if the curved track section is as small as possible. Note that there will be a substantial difference between the profile grade line of the track and the bored tunnel through curves.

Some projects have employed this rotation method to achieve superelevation on aerial structures. Achieving superelevation in this manner can create very complex relationships between the plan and the profile of the track versus that of the structure, particularly on a two-track structure. The twisting of the deck affects the actual profile grade line (PGL) of one or both tracks depending on the point of deck rotation. One project rotated the deck about the low rail of the inside track, resulting in a very large jump in the profile grade line of the outer track through the length of the spiral and an even higher jump in the profile of the outermost rail. Another project rotated the

deck about a structure PGL centered between the tracks and in the plane of the four rails. Note that deck rotation may require the tracks to have identical values of Ea and that the cardinal points of the curves (TS, SC, CS, and ST) on both tracks will need to be directly opposite each other. The vertical component caused by the deck twisting also requires spirals appreciably longer than those normally used since the raising or lowering of each track's PGL effectively induces an additional amount of actual superelevation.

In extraordinary cases, the superelevation may be developed along the tangent preceding the point of curvature (PC), or run off in the tangent immediately beyond the point of tangency (PT). The transition length is then determined from the minimum spiral length formulae presented herein. The maximum amount of superelevation that is run off in tangent track should be no more than 1 inch [25 mm]. Note that this process induces a rotational acceleration that is in the opposite direction from the lateral jerk that occurs when the vehicle enters the horizontal curve, exacerbating the effect of the latter. For this reason alone, introducing superelevation along tangent track is discouraged.

In areas of mixed traffic operation with roadway vehicles, the desired location for a pavement crown is at the centerline of track. Where this is not feasible, a maximum pavement crown of 2.0% (1/4 inch per foot) across the rails may be maintained in the street pavement to promote drainage. This practice will normally introduce a constant actual superelevation (Ea) of approximately 1.2 inches [30 millimeters]. If, at curves, the street pavement is neither superelevated nor the crown removed, this crown-related superelevation may also dictate the maximum allowable operating speed. See Chapter 12 for additional discussion on this issue.

3.2.5.3 Types of Spirals

There are many formulae that either mathematically define or approximate a spiral curve with a progressively varying radius. Types of spirals found in railway alignment design include the AREMA Ten Chord; the cubic spiral; several forms of clothoid spirals as defined by Bartlett, Hickerson, and others; plus various forms of Searles spirals, including some still used by some legacy light rail operations. (Searles spirals are a series of compounded circular curves that approximate the alignment of a clothoid curve.) For the spiral lengths and curvatures found in LRT, all of the above spiral formulae will generally describe the same physical alignment laterally to within ordinary construction tolerances. The choice of spiral easement curve type is thus not critical. It is important, however, to utilize only one of the spiral types and to define it as succinctly as possible. Vague terms such as "clothoid spiral" should be clarified as more than one formula describes this type of spiral curve.

For LRT design, a spiral transition curve that is commonly used in transit work is the Hickerson spiral. Its main advantage is that it is well-defined in terms of data required for both alignment design and field survey work. Figure 3.2.1 depicts a spiraled curve with the associated mathematical formulae as defined by Hickerson.[13]

3.2.5.4 Spiral Transition Curve Lengths

Spiral curve length and superelevation runoff are directly related to passenger comfort. Both the radius and superelevation change at a linear rate through the spiral. The centrifugal force for a given speed is inversely proportional to the instantaneous radius of the superelevation at each

point along the spiral. Thus, lateral acceleration increases at a constant rate until the full curvature of the circular portion of the curve is reached, where the acceleration remains constant until the curve's exit spiral is reached. As a general rule, any speed and transition that provides a comfortable ride through a curve is well within the limits of safety.

Determining easement curve length allows for establishment of superelevation runoff within the allowable rate of increase in lateral acceleration due to superelevation unbalance. Also, the transition must be long enough to limit possible racking of the vehicle frame and torsional forces from being introduced to the track structure by the moving vehicle.

Three parameters must be considered when determining the appropriate spiral length:

- Rate of introduction of unbalance.
- Actual superelevation.
- Rate of change of superelevation.

Depending on the circumstances, one of the three will require a longer spiral and hence govern over the other two criteria. Each of these will be discussed below.

3.2.5.4.1 Length Based upon Superelevation Unbalance

This criterion is fundamentally an issue of passenger ride comfort and controlling the rate at which unbalance (and hence lateral acceleration) is introduced. The steadily increasing lateral acceleration that the passenger feels as the rail vehicle passes through the spiral is aptly known as "jerk," and the pace at which it is introduced is known as the "jerk rate."

As noted previously, the generally recognized maximum acceptable rate of lateral acceleration due to cant deficiency, or superelevation unbalance, for passenger comfort is 0.1 g, where 'g' is the acceleration of gravity, i.e., 32.2 feet per second per second [9.8 meters per second per second]. This pace has been a standard for over a century and was derived empirically based on test observations of trains running at various speeds. It is a conservative value based on average conditions of both rolling stock and track. In the case of track, a design standard of 0.1 g recognizes that the as-built geometry of ordinary ballasted track deteriorates over time and that those incremental deficiencies will collectively result in circumstances where the actual lateral acceleration will be greater than 0.1 g. Hence, a factor of safety is built into the parameter.

However, track geometry is extremely unlikely to deteriorate in direct fixation and embedded trackforms. Short of a significant structural failure, superelevation and horizontal alignment will not change in such rigid track. Therefore, it is possible to allow higher values of lateral acceleration in rigid trackforms. Values up to 0.15 g have been demonstrated to be both safe and comfortable if they are introduced smoothly over the length of spirals of appropriate length. The same value could be used in ballasted track only if the track owner commits to a comprehensive program of track surfacing to maintain track geometry within extremely tight maintenance tolerances. Few, if any, transit authorities have the budget necessary to make that commitment over the long term.

In a curve with no spirals and no superelevation, the lateral acceleration, or jerk, is introduced instantaneously at the point of curvature. Essentially the jerk rate is infinite. Since this is obviously undesirable, the spiral length is usually governed by controlling the jerk rate to a tolerable level.

The preferred formulas presented in Chapter 5 of the AREMA *Manual for Railway Engineering* are based on a maximum rate of change of acceleration of 0.03 g per second. So, if the maximum lateral acceleration is 0.10 g, the spiral should be long enough that a train traveling at the design speed will take 3.33 seconds to traverse it, i.e.:

$$\frac{0.10\,g}{0.03\,g/sec} = 3.33 \text{ seconds}$$

Chapter 5 of the AREMA *Manual for Railway Engineering* allows the jerk rate to rise to an absolute maximum of 0.04 g per second when realigning existing tracks if spiral length is constrained by geographic conditions. However, research associated with the introduction of high-speed passenger rail service in Europe and elsewhere has determined that the jerk rate can be much higher—as high as 0.1 g per second—under controlled circumstances, such as the rigid trackforms noted above. Hence, if both jerk and jerk rate are maximized, the length of the spiral, measured in time, could as little as

$$\frac{0.15\,g}{0.10\,g/sec} = 1.50 \text{ seconds}$$

However, spirals that short should only be employed under extraordinary circumstances after exhaustive investigation has documented that nothing else will work.

Using the more conservative 3.33 seconds for the spiral length, the actual length of the spiral required is 3.33 seconds multiplied by the speed of the vehicle.

Converting to miles per hour [kilometers per hour] the formula may be expressed as

$$L_S \text{ (feet)} = V(mph)(5280/3600) \times 3.33$$
$$= 4.89V \text{ (mph)}$$
$$\left[L_S \text{ (meters)} = V(km/h)\frac{1000}{3600} \times 3.33 \right]$$
$$[= 0.925\,V(km/h)]$$

Assuming that 4 ½ inches [115 millimeters] is the maximum allowable superelevation unbalance, a formula to determine the length of the spiral necessary to ensure passenger comfort can therefore be stated as:

$$L_S = \left(\frac{4.89}{4.5}\right)VE_u \text{ or } L_S = 1.09VE_u \qquad \left[L_S = \left(\frac{0.925}{115}\right)VE_u \text{ or } L_S = 0.008VE_u \right]$$

As a review, the formulae immediately above are based on the parameters stated earlier:

Max Eu = 4.5 inches [115 mm]
Max Jerk = 0.10 g
Max Jerk Rate = 0.03 g/s

By contrast, the preferred formula given in the AREMA *Manual for Railway Engineering*, Ls = 1.63EuV, is based on

Max Eu = 3.0 inches [76 mm]
Max Jerk = 0.10 g
Max Jerk Rate = 0.03 g/s

and the alternate acceptable AREMA formula, Ls = 1.22 EuV, is based on

Max Eu = 3.0 inches [76 mm]
Max Jerk = 0.10 g
Max Jerk Rate = 0.04 g/s

By carefully considering the ramifications of higher values of Eu, jerk, and jerk rate, it is possible to derive even shorter spirals. For example, if lateral acceleration is allowed to rise to 0.15 g (equivalent to 9 inches [230 mm] of unbalance less vehicle roll) and a jerk rate of 0.1 g/s is accepted, the formulae above would become:

$$L_S = \left(\frac{1.71}{9.0 - 1.5}\right) VE_u \text{ or } L_S = 0.29 VE_u \qquad \left[L_S = \left(\frac{0.417}{190}\right) VE_u \text{ or } L_S = 0.002 VE_u\right]$$

As noted above, such extraordinarily short spirals should be used only after extensive investigation and documentation and only in embedded or direct fixation trackforms, where geometric deterioration is virtually impossible. Ordinary alignment work should use either the Ls = 1.09 VEu formula or its S.I. units equivalent.

3.2.5.4.2 Length Based upon Actual Superelevation

This criterion evaluates twist of the vehicle measured over the distance between the trucks. AREMA *Manual for Railway Engineering*, Chapter 5, gives the following formula for determining the length of an easement spiral curve:

$$L_s = 62\ E_a \qquad [L_s = 0.75\ E_a]$$

where Ls is in feet [meters] and Ea is in inches [millimeters]. The only variable in this AREMA formula is the actual superelevation; there's no consideration of speed.

The factor of "62" in the U.S. traditional units version of the equation was empirically derived by one of the AREMA's predecessor organizations based on two considerations:

- 62 feet [19 meters] is roughly the distance between the trucks on a conventional passenger railroad car that is 85 feet [26 meters] long. Observations of such equipment revealed that satisfactory vehicle behavior could be ensured if the difference in track crosslevel from one truck to the other was limited to 1 inch [25 mm] or less.
- "String Lining," the time-honored method for realigning railroad curves, is based on middle ordinate offset distances measured from the outer rail to the midpoint of 62-foot long chords.

Hence, by defining superelevation in terms of 62-foot increments, the AREMA formula used dimensions that were already very familiar to American trackmen. At the time when these

guidelines we developed, much of the field supervision of track construction and maintenance was done by persons who might have had a high school education at most. Hence, unambiguous simplicity was best.

For 6 inches [150 millimeters] of Ea, this AREMA formula produces a spiral 372 feet [113 meters] long. This results in a minimum ratio of superelevation change across truck centers of 1:744. This is an empirical value that accounts for track crosslevel tolerances, car suspension type, and fatigue stresses on the vehicle sills. Also note that the AREMA *Manual for Railway Engineering* formula is applicable to both passenger and freight cars.

Light rail vehicles have a far greater range of suspension travel than freight or intercity passenger cars. The magnitude of the LRV frame twist is relatively small compared to the nominal LRV suspension movement. The maximum actual superelevation runoff rate and minimum ratio of superelevation change across truck centers are thus not fixed values, but are functions of the LRV truck center distance.

The twist-based formula is effectively based on the ability of the vehicle trucks to rotate in a vertical plane relative to the carbody they support. However, truck centers in light rail vehicles are much shorter than in railroad passenger cars. Hence, it is possible to replace the 62 feet in the traditional U.S. units version of the formula with the truck centers of the light rail vehicle. Most light rail vehicles have truck center distances in the range of 25 to 30 feet [8 to 9 meters]. Hence the value of 62 can be replaced by 30. More commonly, a value of 31 is used, half of 62, effectively hearkening back to the time-honored practice of curve string lining. Hence, a traditional formula that appears in many LRT design criteria manuals is

$$L_s = 31\ E_a \quad [L_s = 0.38\ E_a]$$

However, the development of low-floor light rail vehicles with independently rotating wheels has changed the issues. Trucks with solid axles and conventional suspensions are generally sufficiently loose vertically to “equalize” the load on all four wheels when the track is twisting. The new trucks under low-floor cars are not necessarily as limber. It is therefore necessary to consider the short twist between one axle and the next on the same truck. The requirements vary by truck design, but, in general, the builders of low-floor cars require that track twist be limited to an appreciably greater degree than suggested by the traditional formulae above.

A guideline that appears in some European criteria is that twist should not exceed a ratio of 1:400, as in 1 mm of crosslevel difference in 400 mm of track length. That works out to the following version of the equations:

$$L_s = 33.3\ E_a \quad [L_s = 0.40\ E_a]$$

One U.S. transit property, having had appreciable problems with derailments of the center trucks of their partial low-floor LRVs, determined that part of the resolution was to establish a maintenance standard stipulating that superelevation transitions and other track twist situations should be no greater than $^{7}/_{8}$ inch in 31 feet [about 22 mm in 9.45 meters]. That would be equivalent to

$$L_s = 35.4\ E_a \quad [L_s = 0.425\ E_a]$$

However, that threshold is a maintenance standard, not a design and construction criterion. It therefore implies the threshold at which corrective maintenance actions are required and is not a desired design criterion to which the track should initially be constructed.

One very large international carbuilder, so as to accommodate their 100% low-floor LRVs, stipulates that track twist should not result in a difference in gradient between one rail and the other greater than 0.2%. Using that as a guideline, the formulae above become

$$L_s = 41.7\ E_a \quad [L_s = 0.50\ E_a]$$

resulting in minimum spirals about 33% longer than those required by the traditional formula.

At the opposite end of the spectrum are various designs of vintage streetcars, such as operate on many legacy and heritage trolley operations. Data from San Francisco Muni suggest that their heritage PCC cars can reliably negotiate track twist about twice as severe as the traditional formula.

However, while it may be tempting to use such values for a proposed heritage streetcar line, doing so is not recommended. The guideway on any rail transit line is far more permanent than any rolling stock that might run over it. Accordingly, the track alignment designer must anticipate that even if the rail transit service is initiated with rolling stock that is quite limber with respect to twist, it is very likely that some more restrictive vehicle might be used at some future date. A real danger is the possibility that the persons involved in that future vehicle procurement might not realize there is a twist limitation in the track. Sharp horizontal curves are visually apparent; high values of twist are more subtle and hence more likely to be overlooked as an existing condition to which a new LRV must comply.

As a guideline, the following are recommended for defining minimum spiral length as a function of track twist:

Desired minimum (Also, the absolute minimum for LRT tracks shared with freight trains):

$$L_s = 62\ E_a \quad [L_s = 0.75\ E_a]$$

Absolute minimum for systems using 100% low-floor LRVs or which might use such cars in the future:

$$L_s = 41.7\ E_a \quad [L_s = 0.50\ E_a]$$

The formula above can also be considered as an acceptable minimum for systems using only high-floor LRVs with solid axles.

The absolute minimum for systems using high-floor LRVs and which cannot reasonably ever use low-floor cars because of infrastructure constraints (such as train-length high level platforms in subways) would be

$$L_s = 31\ E_a \quad [L_s = 0.38\ E_a]$$

As with all criteria, use of absolute minimums is discouraged, and the track designer should use greater values whenever possible.

Deliberate twist in the track can occur not only in superelevation transitions but also in embedded track whenever the track crosslevel transitions from a normal pavement crown (typically 2%) to a zero cross slope condition, such as might occur in advance of special trackwork. The requisite length of such twist transitions should be calculated in the same fashion as for spiraled superelevation transitions. In cases where the deck of an aerial structure is twisted so as to create a superelevated condition, the deck twisting will alter the profile grade line of the track and create additional actual superelevation in the track. So as to avoid rapid vertical accelerations, this induced superelevation, plus the normal Ea, needs to be factored into the determination of the minimum spiral lengths.

3.2.5.4.3 Length Based upon Both Actual Superelevation and Speed

Prior to 1962, the AREMA (then AREA) *Manual for Railway Engineering* included only one formula for minimum spiral length. It considered how actual superelevation and train speed affected rotational acceleration as the rail vehicle was entering the curve. However, testing during the 1950s revealed that this formula, which ignored superelevation unbalance, could result in spirals with jerk rates in excess of the desired maximum. Because of this, the old formula based on Ea and V was dropped and replaced with those currently in the manual.[8], [9], [10]

A decade later, the Federal Railroad Administration implemented the *Track Safety Standards*, formally known as 49 CFR 213. Among many other things, the FRA standards establish safety criteria for the maximum allowable track twist at various track classes, each class being based on maximum allowable train speed. Track twist can be the result of superelevation transitions, track that is out of crosslevel, or both. Based on the FRA's minimum standards and other factors, each railroad establishes their own criteria for track safety, maintenance, and construction. The construction standards are based on what is achievable when building track so as to provide better than the minimum desired ride quality results at a given speed. The maintenance standards establish a threshold at which corrective action is recommended so as to keep ride quality above a desirable level. Safety standards establish a threshold at which either corrective action or a reduction in train speed is mandatory.

Amtrak has a very comprehensive set of such standards in their field handbook, *Limits and Specifications for the Safety, Maintenance and Construction of Track (MW-1000)*.[16] The values that Amtrak uses for twist in new track construction are based on the FRA track speed classifications. The track class of most interest for purposes of rail transit design is Class 3, which accommodates passenger rolling stock at up to 60 mph [97 km/h]. For Class 3, MW-1000, Subpart C, Paragraph 59.1, requires the design value of twist to be no greater than a ½ inch in 31 feet [13 mm in 9.45 meters]. Plugging those values into a equation in the format of

$$L_s = f\ V\ Ea$$

and solving for "f" results in

$$L_s = 1.03\ V\ E_a \qquad [L_s = 0.0076\ V\ E_a]$$

where

L_s = spiral length in feet [meters]
V = speed in mph [km/h]
E_a = actual superelevation in inches [mm]

In contrast to that, the MW-1000 criteria for Class 9 track (200 mph) allows twist to be up to only a ¼ inch per 31 feet [6mm in 9.34 meters]. However, because of the much higher train speed, that actually allows twist to occur over a much shorter period of time and resolves into the following formulae:

$$L_s = 0.62\ V\ E_a \qquad [L_s = 0.0046\ V\ E_a]$$

The smaller value of "f" in that equation results in shorter spirals than those required by the MW-1000 at slower speeds. This apparent conundrum is because the specified rates of change of crosslevel per length of track are already extremely conservative compared to the FRA safety limits. Use of the more conservative rates could, at extremely high speeds, result in impossibly long spirals.

One European standard[17] (as promulgated by "LibeRTiN," the "*Light Rail Thematic Network*") stipulates that acceptable track twist (including superelevation transitions) can be related to track speed in terms of a ratio in the following format:

$$1{:}10\ V$$

This essentially dictates that the longitudinal distance (in millimeters) necessary to achieve 1 mm of crosslevel is equal to 10 times the velocity (in km/h). This formula is proposed for speeds greater than 30 km/h; at lower speeds a straight 1:300 ratio is proposed. If the LibeRTiN formula is expressed in the format of Ls = f V Ea, substituting 300 mm for Ls, 1 mm for Ea, and 30 km/h for V and converting each of those into feet, inches, and mph respectively, results in a value of f = 1.34. Hence, the LibeRTiN formula can be expressed in the following form:

$$Ls = 1.34\ V\ Ea \quad [Ls = 0.0100\ V\ Ea]$$

where

Ls = the spiral length in feet [meters]

Ea = the actual superelevation in inches [millimeters]

V = train speed in mph [km/h]

As a guideline, the following formulae are suggested for minimum spiral lengths when considering both actual superelevation and speed:

Desired minimum:

$$L_s = 1.34\ V\ E_a \qquad [L_s = 0.0100\ V\ E_a]$$

Acceptable minimum:

$$L_s = 1.03\ V\ E_a \qquad [L_s = 0.0076\ V\ E_a]$$

Absolute minimum:

$$L_s = 0.62\ V\ E_a \qquad [L_s = 0.0046\ V\ E_a]$$

The result should be compared against the minimum spiral lengths defined by the formulae that considered unbalanced superelevation and track twist and the longest spiral selected. Unless Eu has been artificially constrained so as to keep lateral acceleration well under 0.1 g, the formula considering unbalance will usually govern.

As noted in the last paragraph of Article 3.2.5.4.2, the minimum lengths for deliberate track twist situations should be based on the formulae given in this chapter for minimum spiral lengths. Such situations include both changes in crosslevel in embedded track and twisted decks on aerial structures.

In addition to the discussion above, there are a number of documents with good explanations of the derivation of runoff theory; the references at the end of this chapter contain extensive background on the subject.[8], [9], [10], [11]

3.2.6 Determination of Curve Design Speed

The calculation of design speed in curves is dependent on vehicle design and passenger comfort. In addition to the preceding guidelines, curve design speed can be determined from the following principles if specific vehicle performance characteristics are known. This analysis is also necessary if the vehicle dimensions are significantly different than the LRT vehicles described in Chapter 2.

3.2.6.1 Categories of Speeds in Curves

Speed in curves may be categorized as follows:

- Overturning Speed: The speed at which the vehicle will derail or overturn because centrifugal force overcomes gravity.
- Safe Speed: The speed limit above which the vehicle becomes unstable and in great danger of derailment upon the introduction of any anomaly in the roadway.
- Maximum Authorized Speed (MAS): The speed at which the track shall be designed utilizing maximum allowable actual superelevation and superelevation unbalance.
- Signal Speed: The speed for which the signal speed control system is designed. Ideally, signal speed should be just a little faster than the speed at which an experienced operator would normally operate the vehicle so that the automatic overspeed braking system is not deployed unnecessarily.

3.2.6.2 Determination of Eu for Safe and Overturning Speeds

Figure 3.2.4 illustrates a typical transit car riding on superelevated track and the forces associated with the vehicle's center of gravity. Due to the characteristics of the vehicle's suspension system, as it negotiates the curve the center of gravity will shift outboard of a point over the centerline of the track. The resultant vector of the mass of the vehicle and centrifugal force will shift toward the outer rail. A typical high-floor transit car has a center of gravity shift (x) and height (h) of 2.50 inches [63.5 mm] and 50.00 inches [1270 mm], respectively. By contrast, a freight railroad diesel locomotive has typical 'x' and 'h' values of 3 inches [76 mm] and 62 inches [1575 mm], respectively.

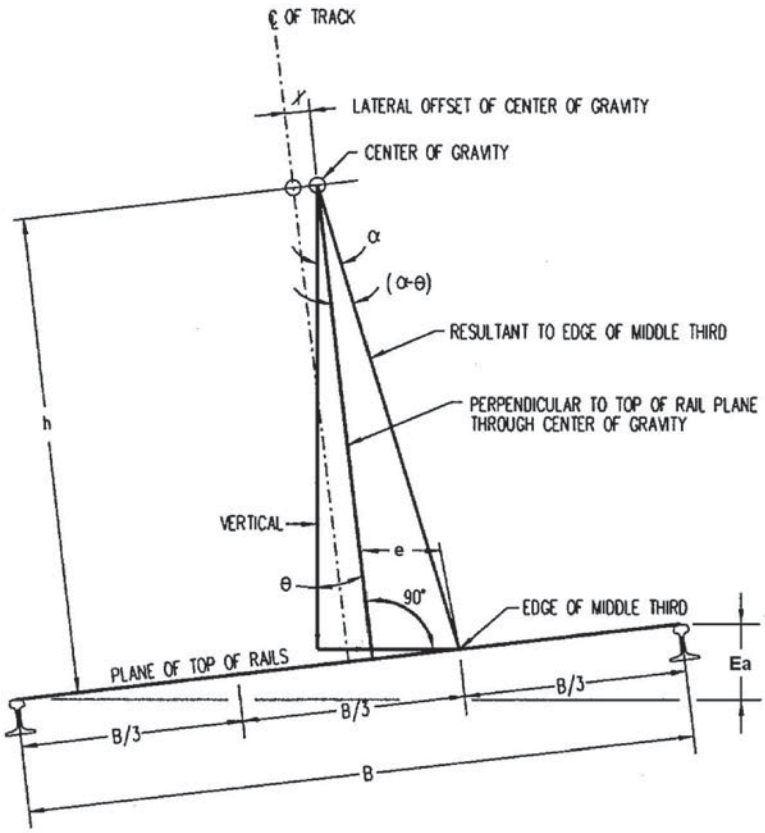


Figure 3.2.4 Force diagram of LRT vehicle on superelevated track

3.2.6.2.1 Overturning Speed

Overturning speed is dependent upon the height of the center of gravity above the top of the rail (h) and the amount that the center of gravity moves laterally toward the high rail (x). When the horizontal centrifugal forces of velocity and the effects of curvature overcome the vertical forces of weight and gravity, causing the resultant vector to rotate about the center of gravity of the vehicle and pass beyond the outer rail, derailment or overturning of the vehicle will occur.

The formula for computing superelevation unbalance for 'Overturning Speed E_u' is derived from the theory of superelevation:

$$\text{Overturning Speed } E_u = Be/h$$

where

B = rail bearing distance = 59.25 inches [1520 mm] as discussed earlier

e = B/2 – x

h = height of center of gravity = 50 inches [1270 mm], which is an average for a typical high-floor LRV

If 'x' = 2 inches [50 mm], then

$$e = [(59.25/2) – 2] = 27.625 \text{ inches} \quad [702 \text{ mm}]$$

then

$$\text{Overturning Speed } Eu = \frac{(59.25 * 27.625)}{50} = 32.7 \text{ inches } [831 \text{mm}]$$

and

$$\text{Overturning Speed } V = \frac{\sqrt{(Eu + Ea) * R}}{3.96}$$

For example, if 'E_a' is given as 6 inches [150 mm] and curve radius is 1145.92 feet [349.3 meters] (a 5^o00'00" curve in arc definition), then

$$\text{Overturning Speed } V = \frac{\sqrt{(32.7 + 6) * 1145.92}}{3.96} = 106 \text{ mph } [170 \text{ km/h}]$$

Obviously, the overturning speed will always be far in excess of the curve's maximum authorized speed.

3.2.6.2.2 Safe Speed

It is generally agreed that a rail vehicle is in a stable condition while rounding a curve if the resultant horizontal and vertical forces fall within the middle third of the distance between the wheel contact points on the rails. This equates to roughly the middle 20 inches [500 mm] of the bearing zone 'B' indicated in Figure 3.2.4. Safe speed is therefore an arbitrarily defined condition where the vehicle force resultant projection stays within the one-third point of the bearing distance. That speed is entirely dependent upon the location of the center of gravity, which is the height above the top of rail 'h' and the offset 'x' of the center of gravity toward the outside rail. From the theory of superelevation, we derive the formula for computing superelevation unbalance for maximum safe speed 'E_u.'

$$\text{Safe Speed } E_u = Be/h$$

where

B = rail bearing distance = 59.25inches [1520 mm])
e = B/6 – x

If 'x' = 2 inches [50 mm], then

e = (59.25/6) – 2 = 7.875 inches [200 mm]
h = height of center of gravity = 50 inches [1270 mm]

then

$$\text{Safe Speed } Eu = \frac{(59.25 * 7.875)}{50} = 9.3 \text{ inches } [237 \text{ mm}]$$

and

Overturning Speed V = square root (((Eu + Ea) x R) /3.96)

For example, if 'E_a' is given as 6 inches [150 mm] and curve radius is 1145.92 feet [349.3 meters] (a $5^o00'00"$ curve in arc definition), then

$$\text{Overturning Speed } V = \frac{\sqrt{(9.3 + 6) * 1145.92}}{3.96} = 66.6 \text{ mph } [107.1 \text{km/h}]$$

3.2.7 Reverse Circular Curves

Where an extremely restrictive horizontal geometry makes it impossible to provide sufficient tangent length between reversed superelevated curves, the curves may meet at a point of reverse spiral (PRS). As a guideline, the PRS should be set so that

$$L_{S1} \times E_{a2} = L_{S2} \times E_{a1}$$

where

E_{a1} = actual superelevation applied to the first curve in inches or millimeters
E_{a2} = actual superelevation of the second circular curve in inches or millimeters
L_{S1} = the length of the spiral leaving the first curve in feet or meters
L_{S2} = the length of the spiral entering the second curve in feet or meters

A separation of up to about 3 feet [1.0 meter] of tangent track between the spirals is acceptable in lieu of meeting at a point of reversal.

The superelevation transition between reversed spirals is usually accomplished by sloping both rails of the track throughout the entire transition spiral, as shown in **Figure 3.2.5**. Note that through the transition, both rails will be at an elevation above the theoretical profile grade line. This method of superelevation transition creates additional design considerations, including an increased ballast section width at the point of the reverse spiral and possible increased clearance requirements. Such issues must be investigated in detail before incorporation into the design.

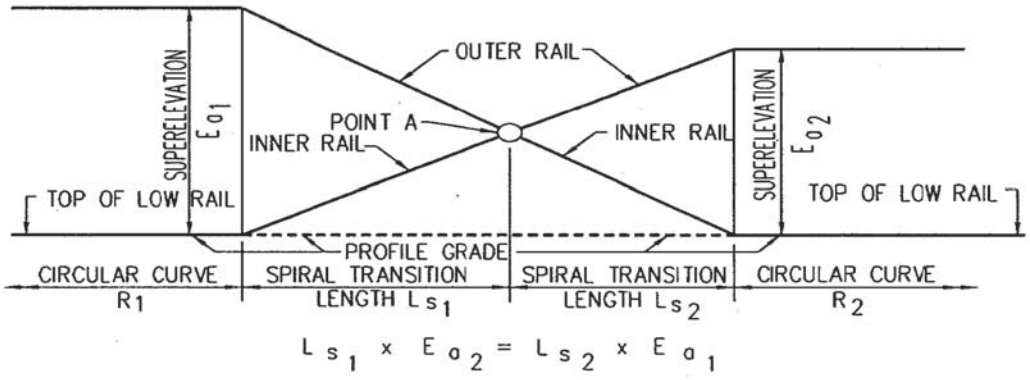


Figure 3.2.5 Superelevation transitions for reverse curves

It is entirely possible to have reverse spirals and remain within acceptable ride comfort criteria. This is indeed the practice for European interurban railway alignments and is occasionally incorporated into North American practice.[6] However, because the direction of lateral acceleration changes at the PRS, the spiral lengths required for reverse spirals to maintain ride comfort should be made appreciably longer than the absolute minimum by limiting the jerk rate,

with 0.03 g/s as a suggested absolute maximum. See Article 3.2.4 for additional discussion on jerk rate and lateral acceleration. Refer to Article 3.2.1 for additional discussion on desirable minimum tangent distances between curves.

3.2.8 Compound Circular Curves

A transition spiral should be used at each end of a superelevated circular curve and between compound circular curves. Between compound curves, the spiral segment, instead of having an infinite radius at one end, will match the radius of the larger curve. The remainder of the spiral between that radius and the theoretical spiral-to-tangent point, where the radius would be infinity, is effectively not used.

The minimum compound curve spiral length is the greater of the lengths as determined by the following:

$$L_S = f1\ (E_{a2} - E_{a1})$$
$$L_S = f2\ (E_{u2} - E_{u1})\ V$$
$$L_S = f3\ (E_{a2} - E_{a1})\ V$$

where

L_S = minimum length of spiral, in feet [meters]

f_1 = the factor used in the corresponding equations for ordinary spiral length based on track twist (i.e., “desirable,” “acceptable,” and “absolute,” minima as appropriate to the design circumstances)

E_{a1} = actual superelevation of the first circular curve in inches [millimeters]

E_{a2} = actual superelevation of the second circular curve, in inches [millimeters]

f_2 = the factor used in the corresponding equations for ordinary spiral length based on unbalanced superelevation and speed

E_{u1} = superelevation unbalance of the first circular curve, in inches [millimeters]

E_{u2} = unbalanced superelevation of the second circular curve, in inches [millimeters]

f_3 = the factor used in the corresponding equations for ordinary spiral length based on actual superelevation and speed

V = design speed through the circular curves, in mph [km/h]

Ride comfort in spiraled compound curves is optimized if Eu is the same value in both circular curve segments.

3.2.9 Track Twist in Embedded Track

When LRT tracks are embedded in pavement and particularly where they are in a shared mixed traffic lane, in many cases the track geometry will be dictated by the roadway agency’s criteria for pavement surface. These are typically dictated by the need to drain storm water off of the pavement surface. As a consequence, there will often be some cross slope in tangent lanes to which the track will need to conform. If this cross slope changes when the street (and track)

enters a curve, twist will occur over some distance. The track designer must verify that this rate of twist does not exceed the criteria specified in this chapter.

It is also important to note that it is unlikely that the street alignment will be spiraled. The spiral lengths in the track must be carefully coordinated with the roadway design so as to both match the pavement surface and keep the horizontal track alignment in an optimal position relative to the traffic lanes. See Chapter 12 for additional discussion on this topic.

3.3 LRT TRACK VERTICAL ALIGNMENT

The vertical alignment of an LRT alignment is composed of constant grade tangent segments connected at their intersection by parabolic curves having a constant rate of change in grade. The nomenclature used to describe vertical alignments is illustrated in **Figure 3.3.1**.

The percentage grade is defined as the rise or fall in elevation, divided by the length. Thus a change in elevation of 1 foot over a distance of 100 feet is defined as a 1% grade.

When using European reference sources, it is fairly common to see gradients defined in terms of the rise or fall in meters per kilometer. This ratio is known as “per mille” (literally, “per thousand” in Latin) and is usually abbreviated as $^{0}/_{00.}$ The similarity between that symbol and the more familiar “percent” symbol (%) can result in much confusion.

The profile grade line in tangent track is usually measured along the centerline of track between the two running rails and in the plane defined by the top of the two rails. In superelevated track, the inside rail of the curve normally remains at the profile grade line, and superelevation is achieved by raising the outer rail above the inner rail. One exception to this recommendation is in circular tunnels, such as might be created by a tunnel-boring machine, In such cases, the superelevation may be rotated about the centerline of track in the interest of minimizing the size of the tunnel without compromising clearances. Note that circular rail transit tunnels follow a different mathematized alignment than the track. The tunnel’s profile grade line (PGL) effectively is coincident with the geometric center of the boring machine. In curved segments, the relationship between the tunnel PGL, the track PGL, and the rails will be complex as the tunnel PGL shifts inboard of the track centerline through curves so that clearances can be maintained.

The vehicle’s performance, dimensions, and tolerance to vertical bending stress dictate criteria for vertical alignments. The following criteria are used for proposed systems using a modern low-floor vehicle. It can be used as a basis of consideration for general use.

3.3.1 Vertical Tangents

The minimum length of constant profile grade between vertical curves should be as follows:

Condition	Length
Main Line Desired Minimum	100 feet [30 meters] or 3 V [0.57 V] where V is the design speed in mph [km/h], whichever is greater
Main Line Absolute Minimum	40 feet [12 meters]

In slow-speed embedded track in urban areas, where the need to conform to existing street profiles makes compliance with the above criteria impracticable, the above requirement is usually waived. Where a tangent between vertical curves is shorter than 40 feet [12 meters], consideration should be given to using reverse or compound vertical curves. This avoids abrupt changes in vertical acceleration that could result in both passenger discomfort and excessive vehicle suspension system wear.

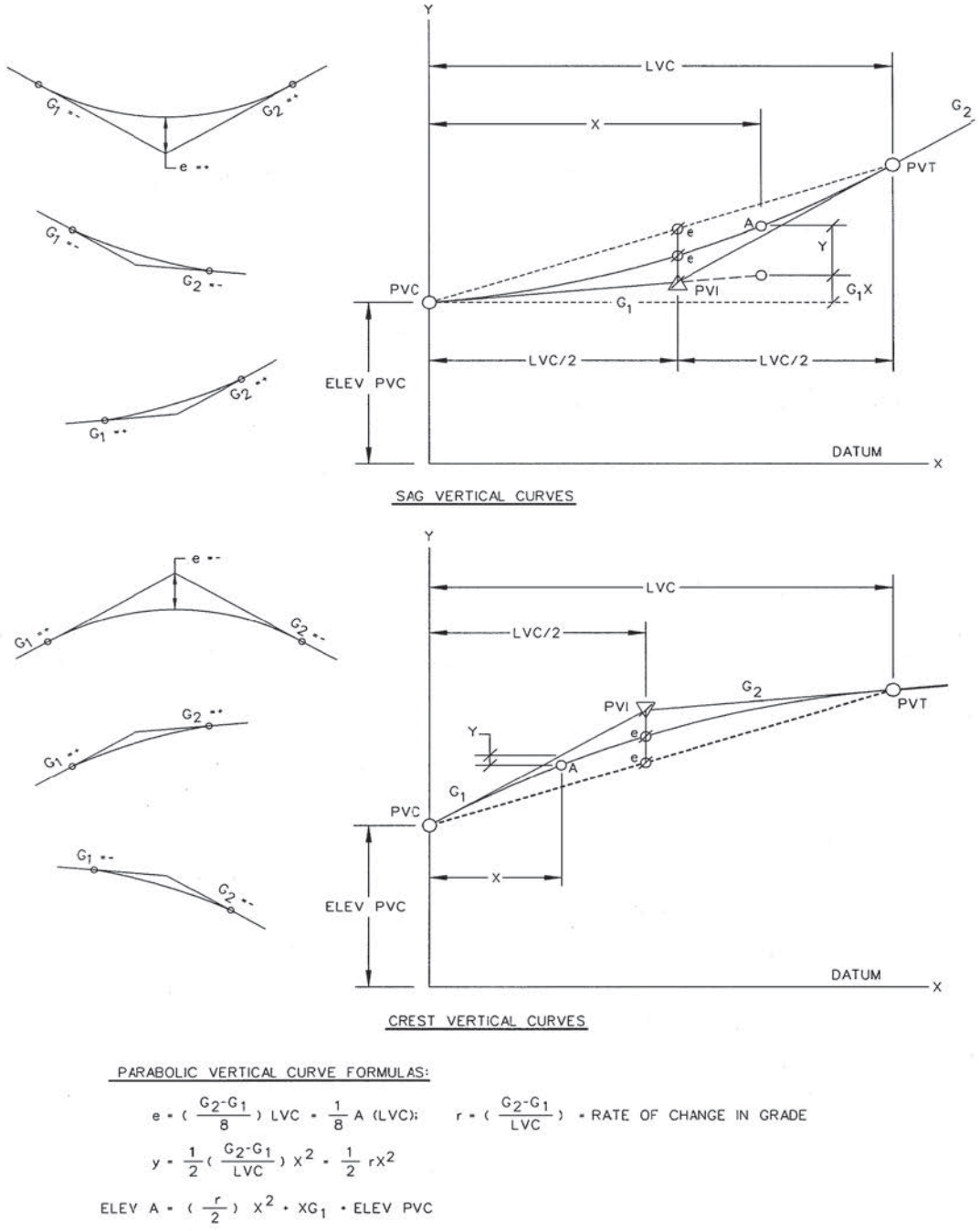


Figure 3.3.1 Vertical curve nomenclature

3.3.2 Vertical Grades

Maximum grades in track are controlled by vehicle braking and tractive capabilities. As explained in Chapter 2, the vehicle capabilities can vary depending on many factors. In addition, because the coefficient of friction between the rail and the wheel can vary depending on environmental conditions, the maximum grade can be affected by the presence of not only water, snow, and ice but also by vegetation, particularly wet and oily fallen leaves. Such rail surface contamination can be a significant issue in embedded track and "grass track."

On main line track, civil drainage provisions often dictate a minimum recommended profile grade. In yards, shops, and at station platforms, there is usually secondary or cross drainage available. Provided adequate drainage can be ensured, tracks that are level or nearly so can be acceptable in ballasted and direct fixation trackforms. See Chapter 4 for additional discussion of trackway drainage.

Embedded tracks need to have some minimum gradient so that not only the pavement surface but also the flangeways will drain. Flangeways accumulate dirt and street debris that needs to be flushed away by storm water runoff. In colder climates, if the flangeways do not drain, there is a possibility of water and debris freezing in the flangeway and causing a derailment. A 2% track grade would be desirable, but may be impractical on many flat urban streets where existing adjoining development prevents any meaningful adjustments in pavement grades.

3.3.2.1 Main Tracks

As a guideline, **Table 3.3.1** provides recommended profile grade limitations for general use in LRT main track design. The desired maximums stated should be acceptable for all light rail vehicles. Some vehicles may be suitable for operation on somewhat steeper "acceptable maximum" gradients.

Table 3.3.1 Maximum and minimum main track gradients

Desired Maximum Unlimited Sustained Grade (any length)	4.0%
Desired Maximum Limited Sustained Grade (up to 2500 feet [750 meters] between points of vertical intersection (PVIs) of vertical curves)	6.0%
Desired Maximum Short Sustained Grade (no more than 500 feet [150 meters] between PVIs of vertical curves)	7.0%
Absolute Maximum Grade Unless Restricted by the Vehicle Design (acceptable length to be confirmed with vehicle designers)	9.0%
Acceptable Minimum Grade for Drainage on Embedded Track	0.5%
Acceptable Minimum Grade for Direct Fixation and Ballasted Trackforms (provided other measures are taken to ensure drainage of the trackway)	0.0%

There are ample examples of grades in existing LRT lines that are both steeper and longer than the desired figures given in Table 3.3.1. For that reason alone, the gradients and lengths above are general guidelines and, within reason, should not be considered as inviolate. For example, there is no compelling reason why a 6.05% grade that is 2,567 feet in length should be automatically rejected. On the other hand, a 6.79% grade that is 3,215 feet in length should be scrutinized more closely, including coordination with the LRV engineers, before being accepted.

Very long hills that incorporate multiple segments with gradients at or near the maximums should also be carefully coordinated with the vehicle engineers. For example, inserting a short segment of 2.0% grade between two segments of 6% grade, each of which individually meets the maximum length criteria, does not necessarily mean that the vehicle won't have issues—for example, the thermal capacity of the friction braking system. Engineering judgment, guided by an interdisciplinary systems approach and considering project and site-specific information, should govern, not arbitrary guidelines such as the figures cited in Table 3.3.1.

On any gradient, tractive forces at the wheel/rail interface (including braking) will always tend to push the rail downhill. Maintaining ballasted track horizontal alignment at the foot of a steep grade is sometimes very difficult, particularly if there is a coincident sharp horizontal curve at that location. Because of this maintenance issue, a rigid trackform (direct fixation or embedded) is preferred for steeply graded tracks. Track designers should consider rigid trackforms for grades steeper than 6%, particularly if combined with sharp curvature and/or frequent hard braking.

3.3.2.2 Pocket Tracks

Where pocket tracks are provided for the reversal of revenue service trains, track grades should preferably not exceed the values stipulated below for yard running tracks. Flatter grades are preferred for pocket tracks since they are often used as temporary storage points for unattended maintenance-of-way equipment and disabled light rail vehicles.

3.3.2.3 Main Tracks at Stations and Stops

See Article 3.5.2 for discussion concerning track gradients at station platforms.

3.3.2.4 Yard and Secondary Tracks

Yard sites are generally preferred to be level so that unattended vehicles cannot roll away. Topography often makes this impractical. In addition, modern transit cars, unlike railroad freight equipment, typically have brakes that are applied by spring action and can only be released by pneumatic or hydraulic pressure. So, as a practical matter, there is little chance that an LRV, parked in ready-for-service condition, might ever roll away. The same cannot be said about vehicles that are either in the shop or stored outside awaiting repair, since their braking systems may be ineffective. Similarly, maintenance-of-way equipment could potentially roll away if parked without hand brakes set. Yards and shop facilities sometimes employ a small locomotive or "car mover" to shift out-of-service vehicles from one track to another. Maximum track grades in the yard should be such that the locomotive's available tractive effort is more than sufficient to move an AW0 light rail vehicle. **Table 3.3.2** provides guidelines that can be used for yard track gradients:

Table 3.3.2 Maximum and minimum yard track gradients

Yard Running Tracks	
Desired	0.5%
Acceptable Maximum	1.0%
Absolute Maximum	Maximum grade for towing or pushing disabled LRVs with the yard's shifting equipment
Yard Storage Tracks	
Desired	0.0%
Acceptable Maximum	0.2%

All tracks entering a yard should either be level, sloped downward away from the main line, or dished to prevent rail vehicles from rolling out of the yard onto the main line. For yard running tracks, a slight grade, usually about 0.5%, is recommended to achieve good track drainage at the subballast level.

Through storage tracks generally have a sag in the middle of their profile to prevent rail vehicles from rolling to either end. Similarly, it is recommended that the profile grade of a stub end storage track descend toward the stub end and, if it is adjacent to a main line or secondary track, it should be horizontally curved away from that track at its stub end. If it is necessary for the profile grade of a storage track to slope up toward the stub end, the grade should not exceed 0.20%.

Tracks located within maintenance shops and other buildings are generally level. However, so that storm water flows away from the building and not into the maintenance pits, it is customary for shop tracks to have a very slight upward slope (typically 0.5% or less) into the building up to the second column line of the building. This distance is typically about 20 to 25 feet [6 to 8 meters]. This gradient would continue across the apron driveway that typically runs around the shop building perimeter.

3.3.3 Vertical Curves

All changes in grade are connected by vertical curves. Vertical curves are defined by parabolas having a constant rate of change in grade. Parabolic curves are, for all practical purposes, equivalent to circular curves for LRT design, but parabolic curves are easier to calculate and are thus preferable for this purpose.

3.3.3.1 Vertical Curve Lengths

The minimum length of vertical curves can be determined as follows:

- Desired Minimum Length: LVC = 200A [LVC = 60A]
- Acceptable Minimum Length: LVC = 100A [LVC = 30A]

- Absolute Minimum Length:

  Crest Curves:

$$LVC = \frac{AV^2}{25} \qquad \left[LVC = \frac{AV^2}{215} \right]$$

  Sag Curves:

$$LVC = \frac{AV^2}{45} \qquad \left[LVC = \frac{AV^2}{387} \right]$$

  where
  LVC = length of vertical curve in feet [meters]
  A = (G_2 – G_1) algebraic difference in gradients connected by the vertical curve, in percent
  G_1 = percent grade of approaching tangent
  G_2 = percent grade of departing tangent
  V = design speed in mph [km/h]

The numerical results from the formulas above are minimums. The designer should use longer vertical curves whenever possible. Both sag and crest vertical curves should have the maximum possible length, especially if approach and departure tangents are long. Vertical broken back curves and short horizontal curves at sags and crests should be avoided.

3.3.3.2 Vertical Curve Radius

As noted in Chapter 2, vehicle manufacturers typically specify a product's vertical capability in terms of either a radius or as a maximum angle that can be tolerated by the articulation joint. Since light rail vehicles are universally designed and built using S.I. dimensional units, these vertical radii are commonly specified in meters. Common figures stipulated by carbuilders (who universally use S.I. units of measurement) for high-floor LRVs for the minimum equivalent radius of curvature for vertical curves located in tangent track are 250 meters [820 feet] for crests and 350 meters [1150 feet] for sags. The track alignment designer must therefore evaluate whether a particular parabolic vertical curve meets the carbuilder's criteria.

This equivalent radius of curvature can be calculated from the following formula, which works in either U.S. traditional units or S.I. units:

$$R_v = \frac{LVC}{0.01(G_2 - G_1)}$$

where

R_v = minimum radius of curvature of a vertical curve in either feet or meters and LVC in the same units

Conversely, the following formula can be used to calculate the requisite vertical curve length given the vehicle manufacturer's criteria for either crest or sag vertical curves.

$$LVC = 0.01\ (G_2 - G_1)\ Rv$$

3.3.3.3 Vertical Curves in the Overhead Contact System

The profile of the contact wire cannot precisely mimic a vertical curve in the track. Instead it is a series of chords with a slight vertical angle at each suspension point with a smoothing of severe trolley grade changes through hanger modifications. Minimum vertical curve length and/or design speed may be governed by the overhead contact system (OCS) due to the maximum permissible rate of separation or convergence between the track grade and the contact wire gradient. Coordination with the OCS designer is strongly recommended to ensure compliance with these limitations.

3.3.4 Vertical Curves—Special Conditions

3.3.4.1 Reverse Vertical Curves

Reverse vertical curves are feasible, provided each curve conforms to the requirements stated in Article 3.3.3 and the restrictions imposed by the LRT vehicle design.

3.3.4.2 Combined Vertical and Horizontal Curvature

Where possible, areas of combined vertical and horizontal curvature should be avoided. Where this is not possible, the track geometry should be as gentle as possible, preferably with neither parameter at or close to a minimum. When extremely constrained site conditions dictate, combined curves should generally not be more severe than an 82-foot [25-meter] radius horizontal combined with a 820-foot [250-meter] equivalent radius vertical crest curve. These parameters must be conformed to the vehicle design specifications.

3.4 TRACK ALIGNMENT AT SPECIAL TRACKWORK

The track alignment must consider the requirements of the special trackwork layouts that will permit tracks to diverge, merge, and cross one another. Users of this Handbook should refer Chapter 6 for guidance on this issue.

The track layout should be supportive of the operating plan, including the location of special trackwork units. In addition to the obvious special trackwork locations, such as junctions and terminal stations, the operating plan should identify locations where emergency crossover tracks are desired so as to facilitate non-scheduled “short turns” movements and temporary single track operations.

When there is a reasonable expectation that an additional branch of the light rail system might be constructed in the future, and the location of the proposed junction can be predicted, it is good design practice to consider the geometric constraints of the future special trackwork in the initial project’s track design. Some projects have even included the construction of the junction needed for the future route in the starter project’s construction. By doing so, it is possible to avoid most of the service disruption that would ensue if the special trackwork installation was deferred.

3.5 STATION PLATFORM ALIGNMENT CONSIDERATIONS

Many of the light rail projects constructed from the 1970s through the 1990s utilized high-floor rolling stock with steps at the doors. Various methods were devised so that riders with disabilities could bypass the steps when boarding and alighting from the trains. In general, those mitigation

measures restricted such passengers to using only one door per train. Use of these specially equipped doors also often required the intervention of the vehicle operator and usually increased station dwell time. However, as the incorporation of ADAAG requirements into projects has advanced, the trend has been toward a strategy of providing level or near-level boarding for all passengers at all doorways of the train. While this course has not been adopted as policy by federal regulatory agencies, it seems likely that level boarding at all doors could be the de facto standard for new start transit projects in the future. With that paradigm as foundation, this article will discuss several issues relative to track alignment at transit station platforms.

3.5.1 Horizontal Alignment of Station Platforms

Tracks through light rail transit stations are preferably horizontally tangent so as to facilitate compliance with the ADAAG requirement for a horizontal gap not greater than 3 inches [75 mm] between the platform edge and the LRV doorway threshold.

So as to minimize the chances that the dynamic envelope might intercept the platform, it is typically necessary to continue this tangent track beyond the end of the platform a minimum distance of one truck center distance plus the vehicle end overhang dimension. This dimension will naturally vary by the vehicle, but 45 feet [13.7 meters] is commonly seen as an absolute minimum in LRT design criteria. Longer dimensions are preferred so that the vehicle suspension system has more time to dampen any carbody roll or translation before the vehicle enters the constrained lateral clearances at the platform. Shorter dimensions are sometimes possible if the vehicle has a significant end taper. The following can be used as general design guidelines for two- and three-section LRVs up to about 90 feet [27.5 meters] long.

Condition	Minimum Tangent Length
Desired Minimum	75 feet [25 meters]
Acceptable Minimum	60 feet [20 meters]
Absolute Minimum	45 feet [15 meters]

For various institutional reasons, it may be necessary to place a station platform in a zone where it is impossible to generate a stretch of tangent track of the preferred length. In such cases, the following options are available:

- The usable platform edge (as opposed to the overall length of platform that is available for passenger queuing) can be limited to the distance from the front edge of the leading door on the first LRV in the train to the back edge of the last door on the last car, plus a stopping tolerance distance. This method can typically shorten the overall length of tangent track required by 30 feet [10 meters] or more. This requires the LRV operator to be more precise about stopping the train so that all doors are on the platform. The platform itself could extend beyond this minimum length but barriers would be required to block access to trackside where the gap is greater than the ADAAG requirement.
- The track through the platform can be placed on a very flat curve—typically no sharper than about 2000 feet [about 600 meters]. This method is often used in conjunction with “sacrificial” thresholds projecting beyond the nominal sides of the light rail vehicle so that any collision causes minimal damage to both the vehicle and the platform edge.

Use of either of these methods requires close coordination with the project architects and vehicle engineers and should be considered for implementation only if extensive study has proven that a full length tangent platform is not possible. Note also that it could become a restriction on the doorway arrangement of any future vehicle procurements.

Stations on sharper radius curves are possible only with gaps that exceed the ADAAG maximum dimension. Some sort of bridge plate would therefore be required to span the gap. This could either be a manually operated device or a device that is automatically deployed when the door opens. However, such arrangements are not recommended for general use. A manually operated device slows down transit operations while it is being deployed, used, and stowed. Further, when the device is not used, the gap will be greater than expected by all passengers and could lead to incidents. Automatically deployed bridge plates have been provided on some LRVs, but are not common. They increase vehicle cost, are still likely to add station dwell time, and complicate the door mechanisms. Doors are frequently one of the least reliable subsystems on any light rail vehicle, and vehicle engineers are understandably reluctant to make them any more complicated than they already are. Those perspectives may change as more experience is gained from current installations.

3.5.2 Vertical Alignment of Station Platforms

Stations should be located on straight tangent grades with a low gradient whenever possible as this simplifies the design and installation of architectural finishes. The following guidance is suggested for track gradients at stations:

- Desirable Minimum: 0.5%
- Acceptable Minimum 0.0%
- Acceptable Maximum 1.0%
- Absolute Maximum: 2.0%

If the track gradient through the station platforms is less than 0.5%, special design measures may be necessary to be certain that the trackway drains. Even if the station is nominally under cover, as it would be in a subway, water will end up on the trackway due to wash water from station janitorial work, precipitation that drips off of the LRVs, and uncontrolled tunnel leakage.

In rigid trackforms, vertical curves can begin immediately beyond the ends of the platform. In ballasted track, the point of vertical curvature should usually be some distance beyond the end of the platform so that any track-surfacing maintenance operations beyond the station can more easily be feathered into the station track profile without affecting the vertical relationship between the platform and the vehicle floor.

When the LRT station is located in a street right-of-way in urban areas, the existing roadway profile will usually govern the profile grade within the station. Sometimes a key station location will fall in a location where the track grade is more severe than the criteria above. While LRT stations have been constructed on gradients as steep as 5%, those installations predate the Americans with Disabilities Act. This creates a potential issue concerning compliance with the ADAAG. While ADAAG permits ramps with gradients up to 8%, they must be periodically interrupted by a landing where persons with disabilities can rest. Such landings are obviously

inconsistent with a platform that follows the track grade. In addition, ADAAG stipulates that paths used by persons using mobility assistance devices such as walkers and wheelchairs should not have a cross slope greater than 2%. A wheelchair sitting facing a track that is on a grade in excess of 2% would hence be a violation of ADAAG. As a result, as of 2010, there is no clear method of having station track grades in excess of 2%. Projects that potentially require stations in constrained urban locations where existing street grades are steeper than 2% will need to work closely with ADA advocacy groups and agencies having jurisdiction to determine if a station is even going to be possible. Such coordination efforts, including documentation of any concessions achieved, should occur as early as possible in the project development process.

While stations are preferably located on straight track gradients, they can and have been constructed on vertical curves as sharp as 2.5% per 100 feet [2.5% per 30 meters]. The platform profile at trackside must be carefully defined so that the vertical step from the platform to the vehicle threshold is within ADAAG criteria. As noted in Chapter 2, it is preferred that passengers have a very slight step downward when exiting the vehicle.

Stations on aerial structures have an additional consideration. If the platform and the track are supported on independent superstructures, their live load deflections could differ substantially. For example, an LRV loaded to AW3 pulling up to an unoccupied platform could, because of the deflection of the superstructure supporting the track, be at a substantially different elevation than the platform, potentially leading to an ADAAG compliance issue. This has occurred on projects where the structure supporting the track was prepared by a different design team than the structure supporting the station. There is nothing the track designer can do about this directly; however, in his/her role as an ad hoc coordinator between disciplines (in this case, the structural engineers and the architects), the track designer can highlight the issue and possibly eliminate a potentially embarrassing issue for the entire design team.

3.6 YARD LAYOUT CONSIDERATIONS

Rail transit yards are very often constructed on oddly-shaped and constrained sites with the result that the track geometrics are unusually complex. The operating plan will typically dictate a routine flow of traffic through the yard, and the track alignment should accommodate this, preferably without requiring reversing movements. For example, there is usually a preferred sequence for what happens when a train comes in off the revenue service route until it is parked in the yard. This sequence could be relatively simple or fairly complex depending on the size and needs of the transit system and when particular daily maintenance activities are performed. The following are the steps for one LRT yard in the northeastern United States:

- The train comes off the revenue service line and proceeds to a cash-handling facility where the fareboxes are emptied.
- The train is advanced to a holding yard where the revenue service operator parks the train.
- A yard hostler picks up the train and runs it through a daily inspection bay in the shop. In addition to inspection of basic issues such as the condition of the wear strip on the pantograph and refilling traction sand boxes, the interior of the car is vacuumed. If necessary or scheduled, the exterior of the car is then washed.

- The hostler moves the car to the main storage yard and proceeds back to the holding yard to pick up another train.

In this case, the yard layout was configured so that all of the activities above could occur while the trains followed a continuous path through the yard, without requiring the hostler to change ends in the vehicle. Other yards will have different sequences depending on the specifics of their operations and maintenance plan. For example, LRT systems located in temperate climates often do car interior cleaning in the main storage yard, after the train has been parked, with the car cleaners carrying their equipment on the equivalent of a golf cart. That methodology requires a different track layout than the example described above, including making the aisles between tracks sufficiently wide to accommodate the golf carts.

If the yard will also be a base for the system's maintenance-of-way (M/W) department, additional tracks will be required for the storage of on-track equipment such as tampers, ballast regulators, overhead line maintenance vehicles, etc. Off-track space should also be provided for the parking of rubber-tired maintenance vehicles, including hy-rail trucks. A location should be provided where hy-rail equipment can get on and off the track. Ideally, the M/W base should have access to the revenue service route without interrupting other yard operations.

Yard layouts can be challenging for the OCS engineer as well as the track engineer because they require consideration of not only the layout of the tracks but also the yard roadway system. The various design disciplines must closely coordinate so as to make certain there are sufficient locations between tracks and also between tracks and roadways so that OCS poles can be installed without requiring special structures.

Because yards are on constrained sites, it is usually necessary to use small turnout sizes. Number 6 turnouts are common in transit yards, and Number 5 and even Number 4 turnouts are not uncommon. Frogs with curved frogs (which technically have no "number") can often be used to good advantage to configure tracks in a tight area; however, it is recommended to avoid special designs unless the overall layout of the yard requires many of them. Since turnouts are involved in a high percentage of derailments, flatter turnouts are always preferred, and it is generally good practice to avoid turnouts with radii that match the minimum curving capability of the vehicle.

The track layout and the layout of the yard's roadway circulation system need to be closely coordinated, and the track alignment engineer is therefore often charged with designing both. The number of track/roadway crossings obviously should be limited, but site constraints make them inevitable. Closely spaced crossings should generally be avoided. The minimum distance between two crossings of the same track should ideally be larger than the longest train so that roadways are not routinely blocked. It is also highly desirable that any roadways within the yard that are routinely used by persons other than transit agency employees (such as outside vendor's delivery trucks) should cross as few tracks as possible and preferably none.

One feature that is very useful in a transit yard is a long stretch of embedded track without OCS. This would become the location where new light rail vehicles can be offloaded from a lowboy tractor trailer. The roadway system should be configured so that these oversized load trucks can

access the embedded track, offload the LRV, and then exit the site, preferably without requiring long backup movements. With delivery of LRVs, the truck leaves the yard complex usually after the teamster has compressed the stretched trailer down to an ordinary legal length. However, the reverse situation also occurs—LRV's being loaded onto a trailer and heading off site, perhaps for a mid-life rebuild at a manufacturer's facility. Accordingly, the roadway system to and from the unloading track should be as flexible as possible.

Additional discussion relative to yard and shop trackwork can be found in Chapters 4, 5, and 6.

3.7 JOINT LRT-RAILROAD/FREIGHT TRACKS

Railroad tracks to be relocated or in joint usage areas are designed in conformance with the requirements of the operating railroad and the AREMA *Manual for Railway Engineering,* except as recommended herein. As a guideline, recommended criteria are given below.

3.7.1 Joint Freight/LRT Horizontal Alignment

The horizontal alignment for joint LRT-railroad/freight tracks consists of tangents, circular curves, and spiral transitions based on the preferred maximum LRV design speed and the required FRA freight class of railroad operation. The track designer will frequently need to consult several criteria documents so as to determine the most restrictive requirements for any given parameter. These would include

- The AREMA *Manual for Railway Engineering* and Portfolio of Trackwork Plans.
- The standard plans and design standards of the freight railroad operator.
- The design criteria, standard drawings, and directive drawings for the LRT project.

As noted previously, railroads usually insist on the use of chord definition for curves and will likely require that for any tracks they will maintain. In addition, it can be expected that freight railroads in the United States will insist that tracks intended for their exclusive use be designed using U.S. traditional units of measurement. References to S.I. units in the text that follows are therefore merely for convenience of reference and metric equivalents have been omitted from the formulae.

The alignment of tracks used by freight trains should preferably be designed for use at not less than 25 mph [40 km/h], which is the FRA maximum freight speed for Class 2 track. When this is not possible, yard track alignment should be designed for an acceptable minimum of 15 mph [25 km/h]. Lead track and industrial sidetracks should be designed for an absolute minimum of 10 mph [15 km/h]. Curves adjacent to turnouts on tracks that diverge from the main track should ordinarily be designed to be no less than the maximum allowable speeds of the adjoining turnouts.

If the existing freight trains in the corridor operate at speeds higher than the above, or could be operated at higher speeds if the physical condition of the tracks was better, it can be reasonably expected that the freight operator will require that existing (or potentially possible) velocities be maintained.

3.7.2 Joint Freight/LRT Tangent Alignment

For joint LRT-railroad/freight main tracks, the desired tangent length between curves should comply with the freight railroad's standards. A desired minimum of 300 feet [90 meters], with an absolute minimum of 100 feet [30 meters], can be used in the absence of more specific guidance. For lead tracks and industrial spurs, a minimum tangent distance of either 60 feet [18 meters] or the longest car using the track should be provided between curve points.

All turnouts should be located on tangents. In general, nothing smaller than a No. 8 turnout should be used unless it is a replacement-in-kind for an existing turnout. No. 10 or larger turnouts are preferred. See Article 3.7.4 for additional discussion concerning turnouts used by freight traffic.

3.7.3 Joint Freight/LRT Curved Alignment

The desired maximum degree of curvature (chord definition) for railroad main line tracks should be either 3 degrees [R = 1910.08 feet/582.193 meters] or the maximum presently in use along the route. As general guidance, main line curves should not exceed 9° 30' [R = 603.80 feet/184.038 meters]. See Article 3.2.3.1 for additional discussion concerning degree of curve as it relates to railroad work. Chord definition should only be used for tracks that will be owned and maintained by the railroad company and then only if they insist upon it.

The maximum curvature for lead tracks and industrial sidetracks should be 12°00' [R = 478.34 feet /145.798 meters]. Larger radii may be appropriate in cases where long freight cars (such as intermodal container cars) use the track. In extreme cases, revisions to existing industrial sidetracks may be designed with curve radii that match the existing values. Exceptions to the above criteria may be permitted as authorized by both the transit authority and the operating freight railroad.

The minimum length of circular curves for main line freight tracks should be 100 feet [30 meters]. Spiral lengths should be as discussed in Article 3.7.6.

3.7.4 Selection of Special Trackwork for Joint Freight/LRT Tracks

Special trackwork in tracks used by freight trains should comply with the standards of the entity that will be responsible for the maintenance of each particular specialwork unit. The reason for this is to simplify maintenance inventory. In joint use tracks, it is typically the transit agency, not the railroad, that will be maintaining the turnouts and hence stocking the spare parts. Conversely, turnouts in freight-only track will typically be maintained by the railroad.

On one shared track LRT project, the freight operator had long before adopted odd-numbered turnouts (e.g., No. 7, No. 9, and No. 11) as their standards. Meanwhile, the LRT system's turnout standards were even numbered (e.g., No. 6, No. 8, and No. 10). The shared main track included several turnouts that led to industries. The track alignment designer used odd-numbered turnouts at these locations even though the transit agency would be maintaining them. The transit agency not only had no spare parts for turnouts of those sizes, they didn't even use the same rail section as the freight operator. Hence, the transit authority maintenance department needed to begin

stocking spare parts for non-standard turnouts even though their LRVs operated over only the straight side of the turnout. Since the freight trains could have easily operated through No. 10 turnouts built using the LRT standard rail section, those maintenance issues could have been avoided by simply using the LRT design at the freight sidetracks.

3.7.5 Superelevation for Joint Freight/LRT Tracks

Superelevation in shared tracks and freight-only tracks should be provided on main line and secondary line tracks only, based on a maximum of 1 ½ inches [38 mm] of unbalance at the freight design operating speed. It will typically be necessary to limit maximum Ea to a range of 3 to 4 inches [75 to 100 mm] depending on the standards of the freight railroad involved. The following assumptions:

- Maximum Ea = 3 inches
- Maximum Eu = 1 ½ inches
- No Ea until Eu has reached ½ inch

result in this equation for determining the preferred value of Ea for the freight speed:

$$E_a = 1.98\left(\frac{Vf^2}{R}\right) - 0.38$$

where

E_a = actual superelevation in inches
Vf = curve design speed for freight traffic in mph
R = radius of curve in feet

As discussed earlier, the calculated values of Ea should be rounded up to the next ¼ inch increment. [Use 5 mm increments when working in S.I. units.]

There sometimes will be a wide divergence between the operating speeds for freight trains versus LRVs. The freight operating speed may also not be consistent, as in cases where freight trains may occasionally be operating slowly in a curve while shifting a nearby industrial sidetrack. A freight speed of 10 mph [16 km/h] would not be unusual under such circumstances. Meanwhile, the LRT operating speed at the same location might be 55 mph [89 km/h]. This may require some compromises, restricting Ea to what the railroad can tolerate and increasing Eu for the LRT to values greater than the customary maximum. Determination of the appropriate spiral length based on all factors is very important.

3.7.6 Spiral Transitions for Joint Freight/LRT Tracks

Spiral transition curves are generally used for railroad/freight main line and secondary line tracks only. Low-speed yard and secondary tracks without superelevation generally do not require spirals. Spirals should be provided on all curves where the superelevation required for the design speed is ½ inch [12 mm].

As a guideline, the minimum length of a spiral in freight-only railroad track and joint use freight railroad and LRT track can be determined from the following formulae, rounded off to the next meter (or 5 feet), but preferably not less than 18 meters (60 feet).

$L_s = 62\ E_a$	$[L_s = 0.75\ E_a]$	(same as AREMA and the desired formula for LRT use)
$L_s = 3.26\ E_u\ V$	$[L_s = 0.018\ E_u\ V]$	(This is based on Eu maximum = 1 ½ inches versus 3 inches in the equivalent AREMA formula.)
$L_s = 1.03\ E_a\ V$	$[L_s = 0.0076\ E_a\ V]$	(same as the desired formula for LRT use)

where

L_s = minimum length of spiral in feet [meters]

E_a = actual superelevation in inches [mm]

E_u = unbalanced superelevation in inches [mm]

V = curve design speed in mph [km/h]

In the case of tracks shared by LRT and freight traffic, the spiral length values calculated from the formulae above must then be compared against the values calculated for the same curves at the proposed LRT speed. The longest dimension will govern.

3.7.7 Vertical Alignment of Joint Freight/LRT Tracks

3.7.7.1 General

The profile grade is defined as the elevation of the top of the low rail. Vertical curves should be defined by parabolic curves having a constant rate of grade change.

3.7.7.2 Vertical Tangents

The absolute minimum length of vertical tangents in joint use track is 100 feet [30 meters]. Turnouts should be located only on tangent grades.

3.7.7.3 Vertical Grades

On main line tracks, the desired maximum grade should be 1.0%. This value may only be exceeded in cases where the existing longitudinal grade is steeper than 1.0%. Grades within horizontal curves are generally compensated (reduced) at a rate of 0.04% per horizontal degree of curvature. Locations where freight trains may frequently stop and start are compensated at a rate of 0.05% per degree of curvature. This compensation reduces the maximum grade in areas of curvature to reflect the additional tractive effort required to pull the train.

For yard tracks and portions of industrial sidetracks where cars are stored, the grades should preferably be 0.20% or less, but should not exceed 0.40%. Running portions of industrial sidetracks should have a maximum grade of 2.5%, except that steeper grades may be required to match existing tracks. Grade compensation is usually not required in railroad yard and industrial tracks.

3.7.7.4 Vertical Curves

Vertical curves shall be provided at all intersections of vertical tangent grades. Length of vertical curves for freight operation should comply with the AREMA *Manual for Railway Engineering*, Chapter 5, Section 3.6. Because of issues associated with the safe operation of freight trains, the AREMA requirements will always result in longer vertical curves than those indicated in article 3.3.4 of this Chapter.

If an existing railroad vertical curve is below the length calculated in accordance with AREMA criteria, a replacement vertical curve with a rate of change of grade not exceeding that of the existing curve may be acceptable at the discretion of the freight railroad.

3.8 VEHICLE CLEARANCES AND TRACK CENTERS

This article discusses the minimum dimensions that must be established to provide minimum clearances between light rail vehicles and adjoining structures or other obstructions and to establish a procedure for determining minimum track center distances.

The provision of adequate clearances for the safe passage of vehicles is a fundamental concern in the design of transit facilities. Careful determination of clearance envelopes and enforcement of the resulting minimum clearance requirements during design and construction are essential to proper operations and safety.

The following discussion concentrates on the establishment of new vehicle clearance envelopes and minimum track centers. On existing LRT systems, this is normally established in the initial design criteria or by conditions in the initial sections of the transit system.

3.8.1 Track Clearance Envelope

The track clearance envelope (TCE) is defined as the space occupied by the maximum vehicle dynamic envelope (VDE) as defined in Chapter 2, Article 2.3, plus effects due to curvature and superelevation, construction and maintenance tolerances of the track structure, construction tolerances of adjacent wayside structures, and running clearances. The relationship between the vehicle and clearance envelopes can thus be expressed as follows:[14]

$$TCE = VDE + TT + C\&S + RC$$

where

TCE = track clearance envelope
VDE = vehicle dynamic envelope
TT = trackwork construction and maintenance tolerances
C&S = vehicle curve and superelevation effects
RC = vehicle running clearance

The clearance envelope represents the space into which no physical part of the transit system, other than the vehicle itself, should be placed, constructed, or allowed to protrude.

A second part of the clearance equation is what is termed structure gauge, which is basically the minimum distance between the centerline of track and a specific point on the structure.

Although structure gauge and track clearance envelope elements are often combined, it is not advisable to construct a track clearance envelope that includes wayside structure clearances and tolerances, as the required horizontal or vertical clearances for different structures may vary significantly.

The factors used to develop the clearance envelope are discussed in further detail in the following sections. It should be noted that in some LRT designs, some of the factors listed above are combined; for example, trackwork construction and maintenance tolerances are frequently included in the calculation of the vehicle dynamic envelope.[2] Regardless of how the individual factors are defined, it is important that all of these items are included in the determination of the overall clearance envelope.

3.8.1.1 Vehicle Dynamic Envelope

Determination of the VDE is discussed in Chapter 2, Article 2.3 as it is typically the responsibility of a project's vehicle design team.

3.8.1.2 Track Construction and Maintenance Tolerances

Track construction and maintenance tolerances should be included in the determination of the track clearance envelope, preferably as a separate item outside of the VDE. This separate consideration is because these track factors will vary depending on the trackform. The track maintenance tolerances are generally far greater than the initial construction tolerances and thus take precedence for the purpose of determining clearances.

It should also be noted that embedded, direct fixation, and ballasted trackwork have different track maintenance tolerances. It is possible to determine separate clearance envelopes for ballasted and direct fixation track or to use the more conservative clearance envelope based on the ballasted trackwork case. Both options have been used in actual practice; however, using a ballasted track clearance envelope for track in a subway could appreciably increase the interior size and hence the cost of the tunnel structure.

Trackwork-based factors to be considered in the development of the clearance envelope, with typical values, include the following:

- Lateral rail wear: ½ inch [13 mm]
- Lateral track alignment maintenance tolerance:
  - Direct fixation and embedded track: ½ inch [13 mm]
  - Ballasted track: 1 inch [25 mm] (Consider larger values for very sharp curves where thermal forces may tend to cause the rail to "breathe" in and out with temperature.)
- Vertical maintenance tolerance:
  - Rail wear: ½ inch [13 mm]
  - Ballasted track settlement/raise: –1 inch / +2 inch [-25 mm / +50 mm]
  - Embedded or direct fixation track slab settlement/heave: As per geotechnical design recommendations.

- Crosslevel variance, direct fixation and embedded track: ½ inch [13 mm] (Largely due to possible temporary differences in rail elevation during future rail changeouts, where one rail might be worn and the other rail new, but also to account for possible differential settlement or heave across the track section.)
- Crosslevel variance, ballasted track: 1 inch [25 mm]

Crosslevel variance creates a condition of vehicle rotation rather than lateral shift. Effects on the clearance envelope are similar to the superelevation effects noted below.

It must be understood that the extreme values suggested above are only for the purposes of determining a track clearance envelope. They are hypothetical worst-case conditions and do not represent thresholds for acceptable maintenance. Similarly, they have nothing to do with the tolerances to be used for construction of new track.

3.8.1.3 Curvature and Superelevation Effects

In addition to the VDE and track maintenance factors, track curvature and superelevation have a significant effect on the determination of the clearance envelope. These effects will be covered separately. Some authorities consider the effects of curvature and superelevation as part of the VDE and calculate separate VDE diagrams for each combination of curvature and superelevation. As a guideline, this Handbook considers only one VDE and determines curvature and superelevation effects separately to establish multiple clearance envelopes.

3.8.1.3.1 Curvature Effects

In addition to the dynamic carbody movements described above, carbody overhang on horizontal curves also increases the lateral displacement of the VDE relative to the track centerline. For design purposes, both mid-car inswing (mid-ordinate) and end-of-car outswing (end overhang) of the vehicle must be considered. While AREMA Chapter 28 includes formulae and tabulated data on clearances, these are generally inapplicable to rail transit vehicles and guideways.

The amount of mid-car inswing and end-of-car outswing depends primarily on the vehicle truck spacing, vehicle end overhang, and track curve radius. The truck axle spacing also has an effect on clearances, although it is relatively small and frequently ignored.[6] Low-floor LRVs with articulation joints that are not centered on the trucks can also measurably shift the position of the end overhang. Collectively, the inswing and outswing and the vehicle's lateral dynamic movements define the edges of what is commonly called the "swept path" of the vehicle. Refer to Chapter 2, Article 2.3.3 for discussion of the vehicle dynamic outline.

To determine the amount of vehicle inswing and outswing for a given curve radius, one of two formulas is generally used, depending on whether the vehicle axle spacing is known. Both methods are sufficiently accurate for general clearance envelope determinations for LRT vehicles.

Figure 3.8.1 illustrates the basic concepts on a hypothetical double-truck rigid car. If truck axle spacing effects are ignored, the effects of vehicle inswing and outswing are determined from the

assumption that the vehicle truck centers are located at the center of track. In this case, the vehicle inswing and outswing can be found from the following equation:

$$\text{Inswing} = M_O = R(1-\cos a) \text{ where } a = \sin^{-1}\left(\frac{L_2}{2R}\right)$$

where

M_o = mid-ordinate of vehicle chord
R = track curve radius
L_2 = vehicle truck spacing

$$\text{Outswing} = R_O - R \text{ where } R_O = \frac{L}{\cos b} \text{ and } b = \tan^{-1}\left(\frac{L}{R - M_O}\right)$$

where

R = track curve radius
L = half of overall vehicle length

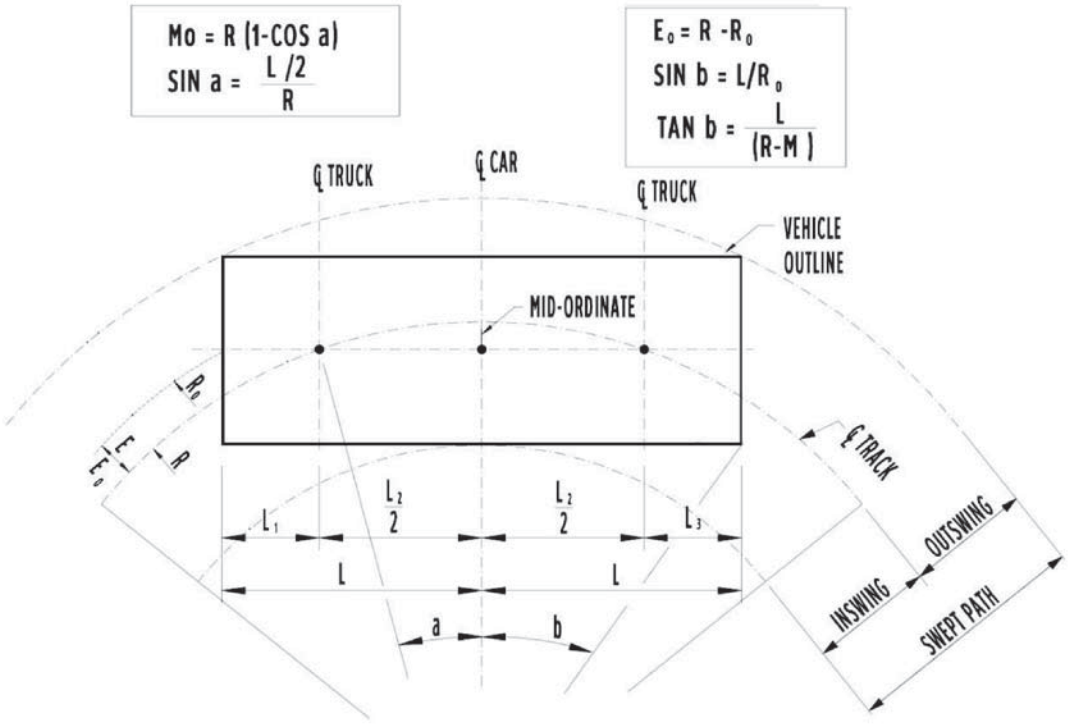


Figure 3.8.1 Horizontal curve effects on vehicle lateral clearance

In determining the outswing of the vehicle, it must be noted that some vehicles have tapered ends and that the outer edge of their swept path will be based on whichever is the worst-case: the vehicle width at the anticlimber or bumper or the full vehicle width at the beginning of the taper. Exterior mirrors on the LRV will often govern outswing, but only at the elevation of the mirror. Hence, the mirror may govern clearances to a wall, but not necessarily to features lower than the mirror. Vehicles that use small cameras as opposed to mirrors will have less impact on outswing

clearance. However, some such vehicles have multiple cameras at strategic points along the side of the vehicle, and one of those might govern inswing at the camera elevation.

When calculating the swept path for horizontal curves with spirals, the tangent clearance envelope will end at some distance ahead of the track tangent-to-spiral (TS) point. The full curvature clearances will similarly begin some distance ahead of the spiral-to-curve point. For an ordinary articulated LRV with two main body sections, these locations can be spotted at one-half the length of the vehicle ahead of the point in question. Typically, this will be about 45 feet [13.7 meters] ahead of the TS and the SC. Between those points, the offsets to the edges of the swept path can be interpolated with sufficient accuracy for most clearance purposes. Similar approximations can be made on simple curves. Where more precise information is required, CADD software makes it relatively easy to graphically determine the edges of the swept path at any location.

The clearance envelope (CE) through turnouts is calculated based on the centerline radius of the turnout.

It is of interest to note that the vehicle designer does not always provide the calculations for the effects of horizontal curvature clearance. This task is frequently left to the trackwork or civil alignment engineer.

3.8.1.3.2 Superelevation Effects

Superelevation effects on the swept path are limited to the vehicle lean induced by a specific difference in elevation between the two rails of the track and should be considered independently of other effects. In determining the effects of superelevation, the shape of the VDE is not altered, but is rotated about the centerline of the top of the low rail of the track for an amount equal to the actual track superelevation.

This rotation is illustrated in **Figure 3.8.2**. For any given coordinate on the VDE, the equations indicated in Figure 3.8.1 are sufficiently accurate to convert the original VDE coordinate (x_T,y_T) into a revised clearance coordinate (x_2, y_2) to account for superelevation effects. Collectively, the effects of all of the factors considered above define the swept path. For convenience, this clearance information is then typically tabulated giving the values of vehicle outswing and inswing for various curve radii and increments of superelevation. **Figure 3.8.3** is a typical example.

3.8.1.4 Vehicle Running Clearance

The clearance envelope must include a minimum allowance for running clearance between the vehicle and adjacent obstructions or vehicles. Running clearance is generally measured horizontally (laterally) to the obstruction, although some clearance envelopes are developed with the running clearance added around the entire perimeter of the vehicle.

The most common minimum value assigned to running clearances is 2 inches [50 mm]. Station platforms are an exception since, per ADAAG, their offset is defined to the static vehicle.

Some items are occasionally assigned a higher minimum running clearance. These include structural members and adjacent vehicles. A typical assignment of running clearance criteria includes the following data:

- Minimum running clearance to signals, signs, platform doors, and other non-structural members: 2 inches [50 mm].
- Minimum running clearance to an emergency walkway envelope: 2 inches [50 mm]. (See note below.)
- Minimum running clearance along an aerial deck parapet, walls, fences, and all structural members, including OCS poles: 6 inches [150 mm]. Note that if a close clearance to a parapet, wall, or fence exists on one side of the track, it is essential that space for personnel to take refuge must be provided on the opposite side.
- Minimum running clearance to adjacent LRT vehicles: 6 inches [150 mm].

Emergency egress safety walkways are located outside of the vehicle clearance envelope. The actual dimensions of the safety walkways are effectively set by NFPA 130, Standard for Fixed Guideway Transit and Passenger Rail Systems.[15] As of 2010, NFPA has increased the recommended sizes of egress paths compared to earlier standards. While dimensions of existing installations may be "grandfathered," transit line extensions and new construction will typically be required to meet the latest standard. Before setting track locations relative to existing structures or setting structure locations relative to new or existing tracks, track designers are advised to work closely with project safety specialists who are thoroughly familiar with the current NFPA 130 requirements.

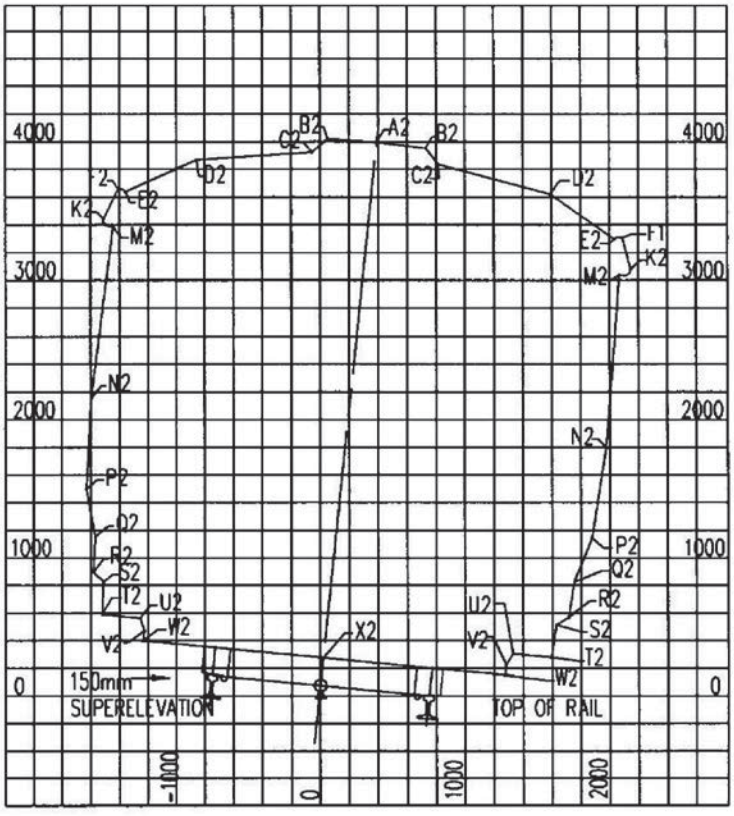


Figure 3.8.2 Dynamic vehicle outline superelevation effect on vertical clearances

Radius (feet)	Superelevation (inches)												
	0.0	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0
	Outswing	Outswing	Outswing	Outswing	Outswing	Outswing	Outswing	Outswing	Outswing	Outswing	Outswing	Outswing	Outswing
82	81.1	80.1	79.2	78.3	77.4	76.4	75.5	74.5	73.6	73.2	72.9	72.6	72.4
95	79.6	78.7	77.8	76.8	75.9	75.0	74.0	73.1	72.1	71.1	70.2	69.2	68.5
100	79.1	78.2	77.3	76.4	75.4	74.5	73.6	72.6	71.6	70.7	69.7	68.7	67.7
150	76.0	75.0	74.1	73.2	72.3	71.3	70.4	69.4	68.5	67.5	66.5	65.5	64.6
200	74.3	73.4	72.4	71.5	70.6	69.6	68.7	67.7	66.8	65.8	64.8	63.9	62.9
250	72.5	71.6	70.7	69.7	68.8	67.9	66.9	66.0	65.0	64.0	63.1	62.1	61.2
300	72.5	71.6	70.7	69.7	68.8	67.9	66.9	66.0	65.0	64.0	63.1	62.1	61.2
400	71.6	70.7	69.8	68.8	67.9	67.0	66.0	65.1	64.1	63.1	62.2	61.4	60.6
500	71.0	70.1	69.2	68.3	67.3	66.4	65.5	64.5	63.5	62.6	61.8	61.0	60.2
550	70.8	69.9	69.0	68.1	67.1	66.2	65.2	64.3	63.3	62.5	61.7	60.9	60.0
600	70.7	69.7	68.8	67.9	67.0	66.0	65.1	64.1	63.2	62.4	61.5	60.7	59.9
700	70.4	69.5	68.6	67.6	66.7	65.8	64.8	63.9	63.0	62.2	61.4	60.5	59.7
800	70.2	69.3	68.3	67.4	66.5	65.6	64.6	63.7	62.8	62.0	61.2	60.4	59.6
900	70.0	69.1	68.2	67.3	66.3	65.4	64.4	63.5	62.7	61.9	61.1	60.3	59.5
1000	69.9	69.0	68.1	67.1	66.2	65.3	64.3	63.4	62.6	61.8	61.0	60.2	59.4
1200	69.7	68.8	67.9	66.9	66.0	65.1	64.1	63.3	62.5	61.7	60.9	60.0	59.2
1250	69.7	68.8	67.8	66.9	66.0	65.0	64.1	63.2	62.4	61.6	60.8	60.0	59.2
1450	69.5	68.6	67.7	66.8	65.8	64.9	64.0	63.1	62.4	61.6	60.7	59.9	59.1
1500	69.5	68.6	67.7	66.8	65.8	64.9	63.9	63.1	62.3	61.5	60.7	59.9	59.1
2000	69.3	68.4	67.5	66.6	65.6	64.7	63.8	63.0	62.2	61.4	60.6	59.8	58.9
2500	69.2	68.3	67.4	66.4	65.5	64.6	63.7	62.9	62.1	61.3	60.5	59.7	58.9
3200	69.1	68.2	67.3	66.3	65.4	64.5	63.6	62.8	62.0	61.2	60.4	59.6	58.8
5000	69.0	68.1	67.1	66.2	65.3	64.3	63.5	62.7	61.9	61.1	60.3	59.5	58.7
10000	68.9	67.9	67.0	66.1	65.2	64.2	63.4	62.7	61.9	61.1	60.2	59.4	58.6
50000	68.8	67.8	66.9	66.0	65.1	64.2	63.4	62.6	61.8	61.0	60.2	59.4	58.5
Tangent	68.7	67.8	66.9	66.0	65.0	64.1	63.4	62.6	61.8	61.0	60.2	59.3	58.5

Basic Assumptions/Criteria:

1. Cross level variation: 1.0 inch.
2. All values are in inches measured from the track centerline.
3. Values worst case calculated values.
4. Maximum vehicle roll angle is 4°.
5. Vehicles are fitted with cameras, not mirrors.
6. Composite envelope derived from the following current production 70% low floor LRVs (KinkiSharyo Hudson Bergen, Bombardier Hiawatha, Siemens 93 ft S70).
7. Calculations include wheel and track tolerances including wheel wear, track wear, rail gauge tolerance, wheel gauge tolerance, nominal sideplay and lateral suspension motion.

Figure 3.8.3 Typical tabulation of dynamic vehicle outswing for given values of curve radius and superelevation

3.8.2 Structure Gauge

The second part of the clearance equation is what is termed structure gauge, which is basically the minimum distance between the centerline of track and a specific point on the structure. This is determined from the TCE above, plus structure tolerances and minimum clearances to structures. Thus:

$$SG = CE + SC + ST + AA$$

where SG = structure gauge
CE = clearance envelope
SC = required clearance to wayside structure
ST = wayside structure construction tolerance
AA = acoustic allowance

The required clearance to wayside structures may be specified separately from the running clearance described above. In other words, the running clearance envelope is stated as a constant value, such as 6 inches, and a separate, additional, required clearance criterion is specified for each type of wayside structure.

Construction tolerances for wayside structures include the construction tolerances associated with wayside structural elements such as walls, catenary poles, and signal equipment. A minimum construction tolerance for large structural elements is normally 2 inches [50 mm]. A larger construction tolerance may be necessary for some types of retaining walls, such as secant pipe walls and soldier pile and lagging walls. It is generally not necessary to include a maintenance tolerance for wayside structures since, unlike track, such items generally are not subject to either wear or post-construction misalignment.

Another item that must be considered is an allowance for chorded construction of tunnel walls, large precast aerial structure sections, and walkways. In lieu of exact construction information, general guidelines that can be used as a basis for design are 50-foot [15-meter] chords for curve radii greater than 2500 feet [750 meters] and 25-foot [7.5-meter] chords for smaller radius curves. See **Figure 3.8.4** for a typical chart of supplemental clearance requirements for chorded construction.

Finally, provisions for present or future acoustical treatments are often required on walls and other structures. Typical values for this range from 2 to 3 inches [50 to 75 mm].

3.8.3 Station Platforms

Station platforms require special clearance considerations because of ADAAG regulations. See Chapter 2 for discussion on this topic.

3.8.4 Vertical Clearances

Vertical clearances are typically set by the collective requirements of the minimum operating height of the vehicle pantograph and the depth of the catenary system. Catenary depth, as discussed in Chapter 11, is the distance from the bottom of the contact wire up to the top of the

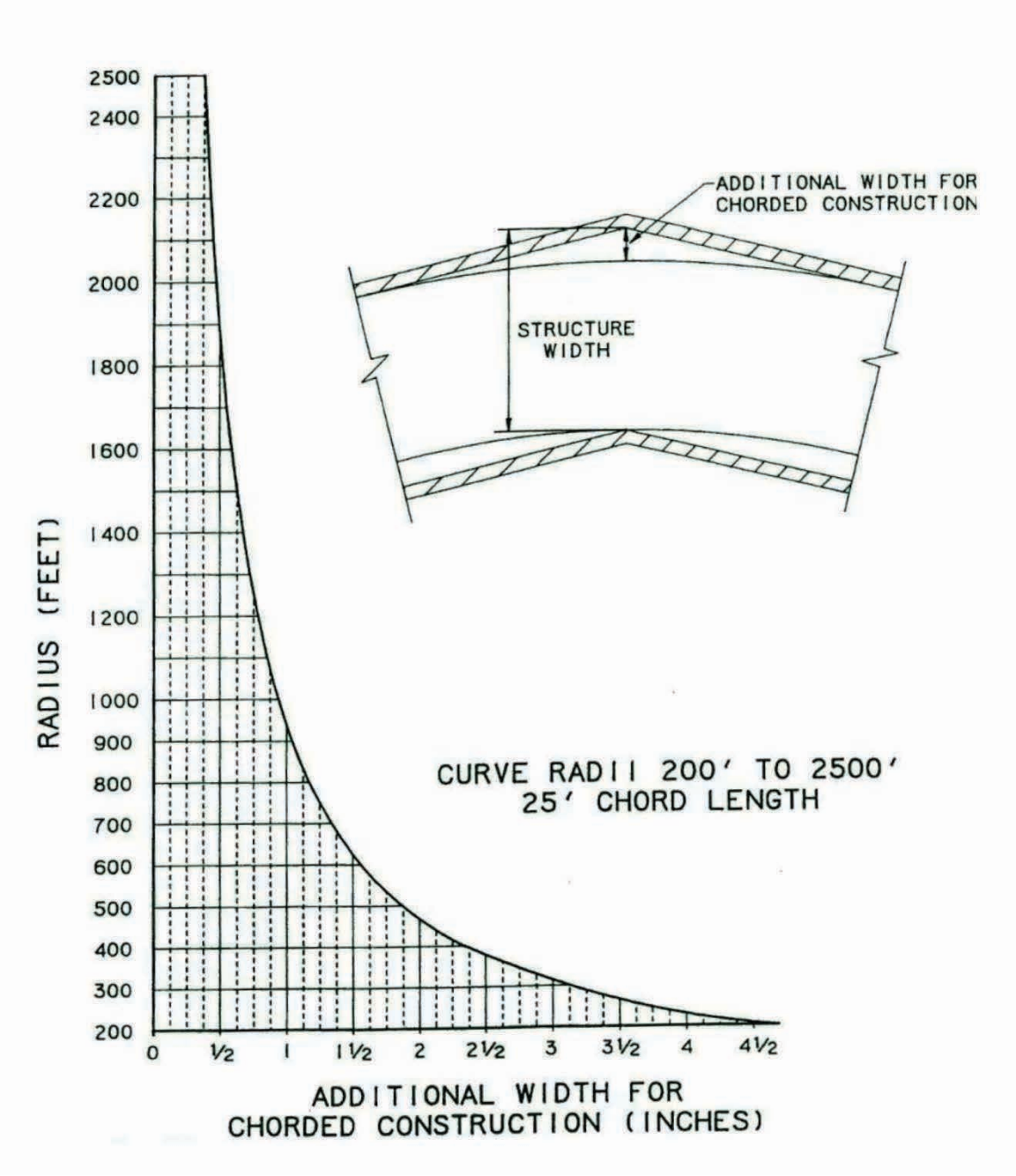


Figure 3.8.4 Additional clearance for chorded construction

support system, plus any required electrical clearances between those supports and adjoining structures.

In ballasted track areas, it is desirable to set vertical clearances to accommodate future track surfacing. Allowances of 4 to 6 inches [100 to 150 mm] are customary. However, the OCS designer will usually want to maximize the depth of the catenary system and he/she and the vehicle engineer will want to maximize the operating height of the pantograph. Therefore, the track engineer may need to defend the track-surfacing allowance from being appropriated by the other disciplines.

Extremely close clearance situations may require using a rigid trackform (e.g., either direct fixation or embedded) or having the authority's maintenance organization commit to track undercutting whenever track surfacing becomes necessary. The design report for the project should specifically address these issues so the project owner understands the options considered and the commitments made.

Because of electrical codes and railroad standards, vertical clearances in shared track areas are far more restrictive than for LRT-only track. Close coordination is required with the OCS designer when setting track profiles in shared track that passes beneath other structures.

3.8.5 Track Spacings

3.8.5.1 Track Centers and Fouling Points

The minimum allowable spacing between tracks and the location of fouling points is determined using the same principles as those used for determining clearances to structures. Referring to the previous discussion on clearances, minimum track centers can be determined from the following equation if catenary poles are not located between tracks:

$$TC = T_t + T_a + 2(OWF) + RC$$

where TC = minimum track centers
T_t = half of vehicle CE toward curve center
T_a = half of vehicle CE away from curve center
RC = running clearance
OWF = other wayside factors (see structure gauge)

Where catenary poles are located between tracks, the minimum track centers are determined from the following:

$$TC = T_t + T_a + 2(OWF + RC) + P$$

where TC = minimum track centers
T_t = half of vehicle CE toward curve center
T_a = half of vehicle CE away from curve center

RC = running clearance
OWF = other wayside factors (see structure gauge)
P = maximum allowable catenary pole diameter

Where the LRT track is designed for joint usage with freight railroads, the clearances mandated by the operating freight railroad and/or state regulatory agencies will prevail. Because railroad employees will occasionally be riding on the side of moving equipment, lateral clearances from the track are usually much greater than for LRT-only tracks. A typical minimum clearance from tangent freight track to any obstruction (such as a catenary pole or signal) is 8'6" [2590 mm]. Some state regulations require even more. The AREMA *Manual for Railway Engineering,* Chapter 28*,* contains useful information on general freight railway clearances, but the individual railroads often have specific clearance requirements that will supersede the AREMA recommendations.

3.8.5.2 Track Centers at Pocket Tracks

Where a pocket track is placed between two main tracks, it is often necessary to provide space for a walkway between the pocket track and one or both of the main tracks. This is because the train operator needs to be able to walk from one end of the train to the other before he/she can run the train in the opposite direction, but LRVs are not typically equipped with end doors that allow direct movement between cars. The walkway typically should not be less than 3 feet (1 meter) wide and should be clear of the swept path on the main track and the static vehicle on the pocket track.

3.8.5.3 Track Centers at Special Trackwork

The track alignment designer must carefully consider the track center distances at any special trackwork layout to make certain the special trackwork can be constructed in accordance with accepted design principles. One such principle is guarding of open frog points. Double crossover tracks are particularly problematic in this regard since the end frogs of the crossing diamond are generally close to being opposite two of the turnout frogs. For standard gauge track, if the track centers are at or close to 14'-0" [4.267 meters], the open throats of the frogs will be virtually opposite each other, making it impossible to guard either point. Unfortunately, 14'-0" is a popular standard track center distance, and this issue has come up on several projects. To mitigate this problem, track centers at double crossovers should be either less than 13'-6" [4.1 meters] or greater than 14'-6" [4.4 meters].

As a general recommendation, whenever a track alignment designer is preparing an area including complex special trackwork, it is strongly recommended that the alignment work and the preliminary trackwork design be done concurrently so that potential problems and issues can be identified before the alignment design is finalized. Doing so will minimize the chance that the alignment might need to be rescinded and revised after it had already been issued to other project design disciplines.

3.9 SHARED CORRIDORS

Where LRT shares a right-of-way (but not tracks) with a freight railroad, the track alignment designer must carefully consider a number of factors when setting horizontal and vertical alignment. These include the following:

- Regulatory Environment: The Federal Railroad Administration generally does not exercise any jurisdiction over rail transit tracks and operations. Exceptions include the obvious case of shared tracks and any tracks that are within 30 feet [9.15 meters] of a track that is part of the national railroad system of transportation. The recordkeeping work of the LRT maintenance-of-way organization could therefore be simplified if the LRT tracks are at least 30 feet [9.15 meters] from the freight tracks.
- Crash Walls: Many freight railroads will insist on a crashwall between their tracks and the transit line if the track-to-track distance is 25 feet [8.7 meters] or less, that requirement being loosely based on AREMA's recommendations concerning crashwalls to protect overhead bridge piers. Notably, the crashwall itself could take a substantial amount of right-of-way width. The issue can sometimes be completely avoided by spacing the tracks no closer than about 26 feet [7.9 meters]. However, some freight railroads have demanded crashwalls even when the separation distance is much greater than 25 feet.
- Ownership of the Right-of-Way: The quality of the title of the real estate occupied by the LRT tracks may be a factor in whether the railroad company can dictate issues concerning the location of the LRT track. If the transit authority purchased property from the railroad, there may be terms in the sales agreement that dictate how the property can be used, including factors related to track location. In some cases, more than one railroad company may use a set of tracks. Depending on the language in legal agreements between the various parties, it may be necessary to meet the minimum standards of both railroads.
- Differences in Track Profile: If the LRT track is at a substantially higher profile than the freight track, but relatively close horizontally, it may be necessary to have retaining walls to support the LRT trackbed, adding substantially to the cost of the LRT construction. On the other hand, some projects prefer to have the transit facility several feet higher than the freight railroad so that, in the event of a freight derailment, railroad equipment is less likely to end up on the transit guideway. The freight railroad may dictate the clearances between the face of the wall and their track.
- Drainage: Both the transit guideway and the freight railroad trackbed will require drainage. Railroads generally dislike closed drainage systems (e.g., underdrains) because they know that such concealed systems have a higher probability of becoming dysfunctional because of neglected maintenance. Hence, the railroad will usually want to have their trackbed drained via open ditches. At the same time, they will not want their ditches used to drain property outside of their right-of-way, including the transitway. Hence, it may be necessary to have two parallel drainage systems—one for the transit line and another for the railroad, particularly if the track profiles are substantially different.
- Right-of-Way Fencing: For various reasons, it may be desirable or necessary to install a fence between the freight railroad and the transit line. There needs to be sufficient space

to install the fence without interfering with either the position or maintenance of other structures, such as drainage systems, train control system signals and bungalows, etc. The fencing will need to be far enough away from each track so as to not interfere with track maintenance activities. Emergency evacuation of the LRVs could be an issue. The freight rail operator may also need safe walking space alongside of the track for train crew members, particularly in switching yards.

- Maintenance Issues: Maintenance-of-way personnel for both the transit agency and the freight railroad need to have access to locations along the guideway and sufficient room to perform their work once they get there. The preferred means of access is an "off-track driveway" usable by maintenance-of-way trucks. In addition, in order to more easily comply with FRA requirements for the safety of their maintenance employees with minimal impact on maintenance productivity, the railroads prefer to have no more than two tracks closely spaced at their standard track center dimension. Looking at the complete cross section of the railroad and transit rights-of-way, this might force the placement of an off-track drive between the two.

It may require far more right-of-way to collectively address the issues noted above than might be apparent at first glance. Notably, decisions about the potential use of shared right-of-way are often finalized during the project planning process, long before many of the topics above are even thought about, much less addressed in any comprehensive manner. At that stage of project development, the track alignment engineer may be one of the few persons on the planning team with any understanding of the physical space requirements that could develop as the project design matures. The track designer should therefore bring these issues to the attention of the project planning staff, carefully evaluate the space requirements, and notify project management should it appear that insufficient right-of-way is being identified to actually construct the infrastructure and systems that will be required.

3.10 REFERENCES

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Chapter 4—Track Structure Design

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CHAPTER 4—TRACK STRUCTURE DESIGN

4.1 INTRODUCTION

The design standards for contemporary light rail transit (LRT) track structures, whether in an at-grade, aerial, or tunnel environment, differ considerably from the principles for either "heavy" rail transit or railroad service. The varied guideway environments in which an LRT system can be constructed result in horizontal and vertical track geometry that often affects light rail vehicle (LRV) design and performance. Consequently, the light rail track designer must consider not only the track geometry, but also the design characteristics of the LRV and how it responds to the guideway geometry. This is particularly true in embedded track located in streets. In general, construction of an LRT guideway in a city street constitutes the greatest challenge to the light rail track designer.

4.2 TRACK AND WHEEL GAUGES AND FLANGEWAYS

The determination of the correct dimensions to be used for track gauge and wheel gauge and for the widths of the flangeways through special trackwork and other guarded portions of the track structure is the most crucial activity to be undertaken during track design. If these design dimensions are not carefully selected to be compatible with the rail vehicle(s) that will operate over the track, unsatisfactory performance and excessive wear of both the track structure and the vehicle wheels will occur.

4.2.1 Vehicle Truck Factors

New, state-of-the-art LRV designs, particularly "low-floor" LRVs, incorporate many features radically different from high-floor LRVs, heavy rail metros, and railroads. These can include smaller diameter wheels, short stub axles with independently rotating wheels (IRWs), and a wide variety of truck axle spacings and truck centers—all of which affect the vehicle's interface with the track structure. In many cases, multiple variations of these factors can occur on a single articulated car. A common situation involves a shorter truck wheelbase on the center non-powered truck of a partial low-floor light rail vehicle. Smaller diameter wheels may also be introduced, and the trams in one European capital city even have two different wheel diameters on the same truck! If these parameters are not carefully considered in the track design, the vehicle's operational tracking pattern can be susceptible to hunting, center truck severe skewing in curves, and unpredictable center truck action at special trackwork. The relationship of track gauge to wheel gauge, particularly the back-to-back ("B2B") dimension between the wheels, is especially important in controlling these operational performance features.

In general, reducing the lateral clearance between the wheel flange and rail head gauge face, either through increasing the wheel gauge (preferred) or decreasing the track gauge, improves wheel tracking of the rail in curves by keeping the truck/wheel as square to the rails as possible. This reduces hunting, skewing, and flange attack angle and results in improved performance through curved track and special trackwork. Vehicle wheel gauge will generally not vary within a given LRV fleet, although cases have occurred where the wheel gauge and wheel profile of a new vehicle procurement have not matched that of the transit agency's existing fleet. It is extremely

important that the track designer take steps to ensure that the vehicle designer does not select wheel parameters independent of track design.

If, as is common, there are several series of vehicles in use on a rail transit system, each with a different combination of truck characteristics, the track designers must consider the worst-case requirements of each car series and optimize the track gauge parameters accordingly.

4.2.2 Standard Track and Wheel Gauges

The majority of contemporary rail transit systems nominally utilize "standard" track gauge of 56 ½ inches [1435 mm]. This track gauge stems from 18th-century horse drawn railways used by English collieries, where track gauge was dictated by the common wheel-to-wheel "gauge" of the wagons used to haul the coal. While many different track gauges were adopted over the years, none have proven to be either as popular or practical as standard gauge.

Track that is nominally constructed to standard gauge can actually be tighter or wider than 56 ½ inches [1435 mm] depending on a variety of circumstances. The track gauge can be adjusted along the route so as to optimize vehicle-to-track interaction. Conditions that can require gauge adjustments include track curvature, the presence or lack of curve restraining rails, and several vehicle design factors. Vehicle factors include wheel diameter, wheel tread taper and width, wheel flange shape including both height and thickness, the distance between axles (also known as "wheelbase"), and the wheel gauge distance between wheels mounted on a common axle.

While nominal 56 ½ inch [1435 mm] standard track gauge is nearly universal for both electric rail transit and "steam" railroads, the different requirements of these modes resulted in appreciably different details, such as where the track gauge is measured, under what conditions it is varied, and the amount of freeplay that is required between the wheel flanges and the gauge faces of the rails.

4.2.2.1 Railroad Gauge Practice

North American railroads set track and wheel mounting gauges in accordance with criteria established by the Mechanical Division of the Association of American Railroads (AAR) and the American Railway Engineering and Maintenance-of-Way Association (AREMA). As shown on AREMA Plan basic number 793, AAR standard wheel gauge is defined as 55 $^{11}/_{16}$ inches [equivalent to 1,414 millimeters] and is measured $^{5}/_{8}$ of an inch [15.9 millimeters] below the wheel tread surface. The AREMA definition of track gauge is measured at the same distance below the top of rail. These gauge standards have been incorporated into many contemporary LRT track designs to accommodate possible joint railroad and LRT operations.

AAR promulgates two wheel profiles. The AAR-1B Narrow Flange wheel is designed for locomotives and passenger equipment. The AAR-1B Wide Flange wheel is intended only for freight cars. If wheels using the AAR-1B Narrow Flange wheel are mounted at standard AAR wheel gauge and the wheel and axle assembly is centered between the rails at standard track gauge, the horizontal clearance between the wheel and the rail at the gauge line elevation is $^{13}/_{32}$ inch [10.3 mm] as shown in **Figure 4.2.1**. This results in total freeplay between correctly mounted (and unworn) wheelsets and exactly gauged rails of $^{13}/_{16}$ inch [almost 21 millimeters].

For trucks with conventional solid axles and not independently rotating wheels, the freeplay assists in the steering or curving of the axle by the differential in wheel diameters, provided the wheel treads are tapered. See Article 4.2.4.1 for additional discussion on this point.

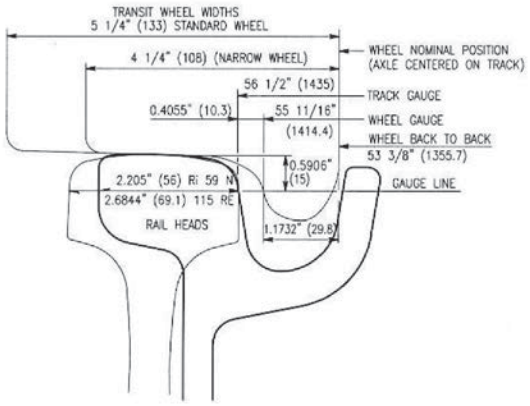


Figure 4.2.1 AAR-1B narrow flange wheel (superimposed on 115 RE and 59R2 rails)

It is important to recognize that railroad gauge practices generally evolved in a different environment than transit operations. Particularly for curved tracks, railroad criteria are predicated on the use of equipment that generally has much larger diameter wheels than those used on transit vehicles. In addition, both the maximum wheelbase and the number of axles that might be mounted on a rigid truck frame are usually much greater. Steam locomotives in particular could have wheels over 6 feet [1.8 meters] in diameter, with up to five such sets of wheels on a rigid frame. Even contemporary diesel locomotives can have wheels that are 42 inches [almost 1.1 meters] in diameter, with three wheel and axle sets on trucks that can have an overall wheelbase of 13 feet [nearly 4 meters]. By contrast, contemporary rail transit vehicles rarely have wheels over 28 inches [711 mm] in diameter, never have more than two axles per truck, and generally have maximum wheelbase distances no longer than 6.00 to 6.25 feet [1800 to 1900 mm]. Only one U.S. LRT system has a longer wheelbase, and it occurs on a unique vehicle design that is unlikely to ever be duplicated.

The much larger truck features associated with railroad equipment dictate relationships between wheel gauge and track gauge that are far less constrained than those required for transit equipment. In addition, freight car wheel maintenance tolerances both for wheel contour and back-to-back ("B2B") wheel gauge are far looser than those of insular transit systems. Freight track must therefore be more forgiving. Hence, it is recommended that railroad track gauge, wheel gauge, and flangeway width criteria not be adopted for an LRT track system unless both transit and freight railroad equipment will operate jointly on a common track.

4.2.2.2 Transit Gauge Practice

Traditional street railway/tramway systems developed guidelines for wheel gauge that differ considerably from guidelines used by railroads. In the United States, the most common standards for track and wheel mounting gauges were those promulgated by the American Electric

Railway Engineering Association (later renamed the American Transit Engineering Association or ATEA). The ATEA standard track and wheel gauges were 56 ½ and 56 ¼ inches [1,435 and 1,428 millimeters], respectively, and were measured ¼ inch [6 millimeters] below the top of the rail. In addition, some transit systems tightened the track gauge in tangent track, taking advantage of a compound curve gauge corner radius that was rolled into the head of some ATEA girder rails. The few "legacy" North American light rail systems that predate the 1970s renaissance of light rail transit typically follow wheel gauge standards that can be traced back to ATEA recommendations. European tramways developed similar standards, although it is important to note that, in general, European street railways use wheel flanges that are even smaller than those promulgated by ATEA.

The transit type standards for wheel gauge have several advantages:

- With a tighter gauge relationship, truck hunting—the lateral oscillation of a truck from one rail to the other as it seeks a consistent rolling radius on all wheels—is more easily controlled. Hunting typically is a tangent track phenomenon and is more prevalent at higher vehicle speeds. Hunting has multiple causes, including the spring rate of the truck's primary suspension.
- Trucks cannot become as greatly skewed to the track, thereby reducing the angle of attack between the wheel flange and the gauge face of the rail (also known as "flange bite") in tangent and curved track.
- Flangeways can be appreciably narrower, a significant consideration for embedded track areas with significant pedestrian activity. This coincidently permits the use of groove rails with relatively narrow flangeways when desired.

Generally, tight wheel-gauge-to-track-gauge relationships can only be employed when the transit operator does not have to share its tracks with a railroad. There are exceptions in Europe where the transit systems have implemented special designs of wheels and special trackwork to permit "tram-train" LRT operations. These systems use tramway tracks in city streets and switch to freight railroad tracks in suburban areas. The first such operation was in Karlsruhe, Germany, and several other transit systems have implemented similar services.

Most North American LRT systems do not share track with freight railroads. Since they are thus not restricted by AAR practices, they feature a wide variety of vehicle wheel profiles and gauges even though most employ standard track gauge of 56 ½ inches [1435 millimeters].

As a guideline, **Figure 4.2.2** illustrates a suggested wheel gauge for transit use with standard track gauge of 56 inches [1422.4 millimeters]. Use of this wheel gauge results in ½ inch of total freeplay, which is effectively a compromise—$^{5}/_{16}$ inch [8 mm] less than AAR wheel gauge practice but ¼ inch [6 mm] more than the freeplay endorsed by the former ATEA. The freeplay between each wheel and the rail it is riding on is therefore ¼ inch [6.35 millimeters]. Readers should compare Figure 4.2.2 against Figure 4.2.1 to see the differences between railroad and transit wheel gauge practice. In particular, note that the transit wheel illustrated in Figure 4.2.2 uses a thinner flange than the AAR- 1B wheel. Because of this difference, the B2B dimension on the transit wheelset is ⅞ inch [22 mm] larger than AAR practice. The combination of thinner flanges

and a larger B2B dimension is what permits LRT operations to successfully use groove rails with narrow flangeways.

See Article 4.2.2.5 for additional discussion related to freeplay and the use of narrow flangeway groove rails.

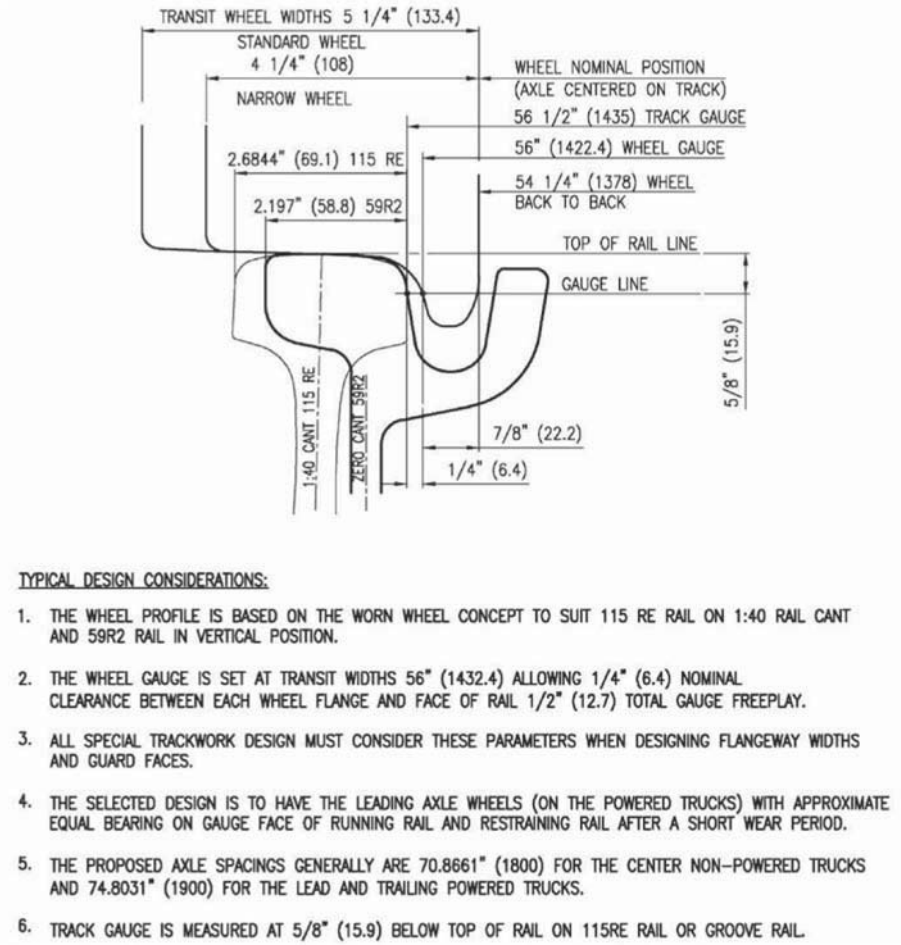


Figure 4.2.2 Suggested standard wheel gauge—transit system

4.2.2.3 Gauge Measurement Location

Track gauge is measured a specific distance below top of rail because of the gauge corner radii of the rail and the flange-to-tread fillet radius of the wheel. The location where gauge is measured frequently differs between railroad and transit systems. The customary gauge elevation point on North American railroads is $^{5}/_{8}$ inch [15.9 millimeters] below top of rail. Track gauge on traditional street railway systems was, and in some instances still is, measured at either ¼ inch [6.4 millimeters] or $^{3}/_{8}$ inch [9.5 millimeters] below top of rail.

Rail sections with compound gauge corner radii, such as 115 RE section (see **Figure 4.2.3**), do not have a nominally vertical tangent section for gauge measurement at the ¼-inch [6.4-mm] or $^{3}/_{8}$-inch [9.5-millimeters] height, hence the designation of a lower elevation. Older rail sections that were prevalent when the ATEA promulgated its standards, such as ASCE, ARA-A and ARA-

B rail sections, had smaller gauge corner radii and thus were more conducive to gauge measurement closer to top of rail. Such rail is no longer commonly rolled in North America. Since measurement of gauge within the curved portion of the rail head is difficult at best and misleading at worst, it is recommended that gauge elevation be defined consistent with railroad practice. North American transit systems should therefore designate gauge elevation at 0.625 inches [15.9 millimeters] below top of rail for tee rail.

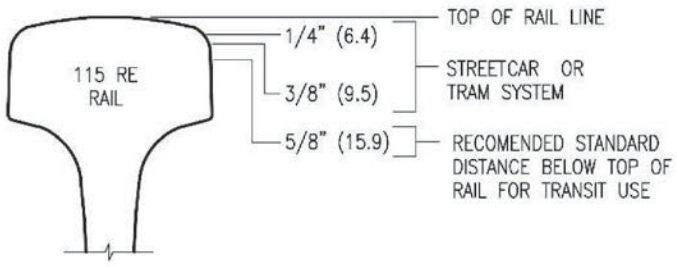


Figure 4.2.3 Gauge line locations on 115 RE rail head

European practice for gauge line elevation ranges from 10 to 15 millimeters [0.39 to 0.59 inches] depending on the source of the information. A gauge line elevation of 10 millimeters [about $^{3}/_{8}$ inch] is inappropriate simply because it is still within the gauge corner radius of the rail head. Moreover, any differences between 15 mm and $^{5}/_{8}$ inch would be totally masked by ordinary fabrication and construction tolerances. The researchers believe that for systems using modern rail sections with compound gauge corner radii, such as 115 RE tee rail and 60R2 groove rail, the North American convention of $^{5}/_{8}$ inch [15.9 millimeters] is an appropriate elevation for measuring track gauge.

4.2.2.4 Gauge Issues—Joint LRT and Railroad and Mixed Fleets

For a system with a mixed fleet, compromises may be required to accommodate a variety of truck and wheel parameters. This problem is not new—early 20th-century electric street railway track designers frequently had to adapt their systems to handle not only city streetcars with short wheelbase trucks and relatively small diameter wheels, but also "interurban" trolleys that typically had longer wheelbase trucks and larger diameter wheels. Some trolley companies even offered freight service and routinely handled "steam" railroad freight cars over portions of their lines. Today, if a light rail system shares any portion of its route with a freight railroad, or if future extensions either will or might share freight railroad tracks, then conformance with freight railroad gauge and other freight geometry constraints may control some elements of the track design.

When a new light rail system shares track with a freight railroad, freight operations normally occur only along ballasted track segments. It is unusual for freight trains to share aerial structure or embedded track segments of a system. In general, the mixing of rail freight and LRT operations on any portion of a system will govern track and wheel gauge design decisions for the entire system unless Karlsruhe-type compromise wheels and special trackwork designs are adopted. Compromises will be required both on the vehicle and on the shared track and may have some effect on the transit-only portion of track on the same system as well. Even if the system's "starter line" does not include joint operation areas, consideration should be given to whether future extensions of the system might share tracks with a freight railroad.

Regardless of whether or not joint operation with a freight railroad is contemplated, there are several key issues to consider. These include the setting of the back-to-back wheel dimension, guard check gauge, and guard face gauge criteria that result from a particular wheel setting. Track design parameters that will be most affected by these decisions include

- The practicality of using available groove and guard rails that are rolled with a specific flangeway width.
- The flangeway width and track gauge required for effective restraining rail or guard rail applications.
- Details for guarding of frog points (both turnouts and crossing diamonds) in special trackwork locations.

Transit systems that do not share tracks with a freight railroad may still have a track connection at the maintenance facility yard for delivery of freight cars loaded with track materials or the system's new light rail vehicles. If the system's maintenance plan contemplates movement of railroad rolling stock (such as hopper cars full of ballast) over portions of the system, it may be necessary to compromise the track design to accommodate the railroad equipment. This does not mean wholesale adoption of railroad standards. Provided that the guard check gauge at turnout frogs allows sufficient space for AAR back-to-back wheel gauge, freight cars can usually be moved over open track portions of an LRT system at low speeds. It may be necessary to prohibit any railroad equipment with wheels that are not precisely mounted, as maintenance tolerances for railroad wheel settings are considerably more liberal than those applied to rail transit fleets.

AAR standard wheel profiles and wheel gauge on railroad equipment is a very important issue when considering occasional operation of railroad equipment over a track system designed for LRT-only service. Embedded track areas that utilize narrow flangeway groove rails typically cannot accommodate movements of railroad rolling stock through curves with radii less than about 300 feet [approximately 91 meters]. Groove rails with wide flangeways that can accommodate freight rolling stock are available, but the flangeways are wider than desirable. See Chapter 5 for additional information. Other restrictions on railroad equipment movements involve the structural capacity of bridges designed for LRT loads and clearances to trackside obstructions such as catenary poles and station platforms.

Another category of joint operations is where it is proposed to extend an existing "heavy" rail transit operation using light rail technology. The existing system will already have track gauge, wheel gauge, and wheel profile standards in place that must be considered in the design of the light rail tracks and vehicles for the new system. If the truck parameters of the existing rolling stock, such as truck wheelbase or wheel diameter, are appreciably different from typical LRV designs, compromises will be necessary to achieve compatible operations. Special consideration must be given to existing maintenance-of-way vehicles, such as hy-rail trucks, since their wheel profiles and mounting dimension may be inconsistent with the new extension's track design.

Even if neither railroad rolling stock nor mixed transit car fleets are a consideration, the trackwork designer should consider the ramifications that track and wheel gauge variations might have for on-track maintenance-of-way equipment. It is imperative that specific notification be given that

the transit system's gauge standards differ from AAR and AREMA standards so that construction and maintenance equipment do not damage the track. Refer to Chapters 13 and 14 for more on this subject.

4.2.2.5 Gauge Issues for Embedded Track

The appropriate track gauge to use in embedded track is highly dependent on the rail section (either tee rail or groove rail) and the vehicle wheel gauge. In this regard, it is very important to note that standard railroad wheel contours (e.g., AAR-1B) and railroad wheel mounting gauges are not compatible with narrow flangeway groove rails presently available from European mills if the track is built to 56-½-inch [1435-millimeter] gauge. The backs of the wheels will bind with the tram or guarding lip of the groove rail causing one flange to ride up out of the flangeway. If narrow flangeway groove rails—such as 51R1, 53R1, 59R2, and 60R2—are selected, it will be necessary to adopt either a wide wheel gauge or an equivalent narrow track gauge. Narrowing the overall track gauge to something less than standard was occasionally employed on legacy rail transit systems, but is no longer a common practice. It could be considered under extenuating circumstances, but, in general, it is not recommended due to the impact on all equipment required to maintain the track system.

Embedded track is typically separated from joint use track. However, if railroad standard wheel gauge must be employed on an LRV because some portion of the route shares track with a freight railroad, wheel clearance to the embedded groove rail track can alternatively be achieved by reducing the track gauge only in those areas where the groove rail is installed. This will reduce the wheel-rail clearance at the gauge line ("freeplay"), alter the rail/wheel interface compared to other portions of the route, and may result in unsatisfactory interaction with both transit and railroad equipment. It may be possible to mitigate these issues by adopting special rail-grinding profiles in any areas of tightened gauge; however, note that rail grinding in embedded track areas is more difficult in any event. Railroad equipment movements that are limited to occasional maintenance work trains at low speed may be acceptable. The above measures should only be considered after detailed study. Also, note that the track designer will have no control over the condition of the wheels of any freight equipment that operates over nominally LRT-only track. The track designer cannot safely assume that operations and maintenance personnel responsible for any such possible future movements will diligently scrutinize the condition of the wheels of any interchange equipment and reject those that do not comply with some standard higher than AAR's interchange rules.

If routine joint operation with railroad freight equipment along an embedded track area is expected, use of narrow flangeway groove rails will not be possible. Wide flangeway groove rails for freight railroad use are provided by some European rolling mills, but, presently, available designs of this type have flangeways that are so wide and tram height that is so low that they cannot provide any appreciable guarding action for curves or special trackwork. This was not the case with girder guard rails made in North America until the mid-1980s; however, these rails can no longer be obtained. A near match of the head and flangeway contours of the former North American designs can be achieved by milling the head of one of the structural groove rail sections available from European mills; however, this is an expensive solution that requires careful investigation and justification. See Chapter 5 for discussion of procurement issues related to European groove rails.

More latitude for joint operations in embedded track can be achieved using tee rails rather than groove rails; however, a separate flangeway must be constructed and maintained in the pavement surface. Refer to Chapter 5 of this Handbook for additional discussion concerning the application of tee rails to embedded track.

4.2.2.6 Non-Standard Track Gauges

In addition to standard 56-½-inch [1,435-millimeters] track gauge, several other gauges have been used on light rail transit systems in North America and overseas. Narrow gauge systems, typically meter gauge [39.37 inches], are relatively common in Europe, particularly in older cities where narrow streets restrict vehicle sizes. There were once many narrow gauge street railways in North America; however, the only survivors are the San Francisco cable car system and a trolley museum near Los Angeles.

Broad gauge trolley systems were more common, and, for a period back in the 1960s and 1970s, there were actually more miles of broad gauge streetcar track in North America than there were standard gauge trolley lines. Four legacy streetcar operations in North America use broad track gauges. These range from 58 $^{7}/_{8}$ inches [1,496 millimeters] in Toronto to 62 ¼ inches [1,581 millimeters] in Philadelphia and 62 ½ inches [1,588 millimeters] on the Pittsburgh and New Orleans systems. Such odd gauges were typically dictated by the municipal ordinances that granted the streetcar companies their "franchise" to operate within the city streets. In such legislation, it was typically specified that the rails should be laid at a distance apart that conformed to local wagon gauge, thereby providing horse drawn wagons and carriages with a smoother running surface than the primitive pavements of the era. The only new start transit operation in North America to adopt a non-standard track gauge in recent years was San Francisco's BART "heavy" rail system at 66 inches [1,676 millimeters]. This gauge was reportedly intended to provide increased vehicle stability against crosswinds for a proposed but never built bridge crossing of San Francisco Bay.

Those systems that employ unusual gauges typically rue the fact because it complicates many facets of track and vehicle design, construction/fabrication, and maintenance. Contracting for services such as track surfacing and rail grinding becomes more difficult and expensive since contractors do not routinely have broad gauge equipment on hand and converting and subsequently reverting standard gauge equipment for a short-term assignment is time consuming and expensive. Vehicle procurement is also complicated since off-the-shelf truck designs must be modified, and potential savings from joint vehicle procurements cannot be realized. Wide gauges also preclude joint operation of a rail transit line on a railroad route since dual gauge special trackwork and the train control systems necessary to operate it are both extremely complex and expensive. Accordingly, non-standard gauges are not recommended for new start projects. Systems which presently have broad gauge track most likely need to perpetuate that practice for future extensions so as to maintain internal compatibility in both track and rolling stock design. Notably, Toronto's "Transit City" LRT expansion program is utilizing standard track gauge as it has no interface with their legacy streetcar system.

4.2.3 Track Gauge Variation—General Discussion

Light rail transit tracks that are constructed with conventional tee rails and operate only light rail vehicles with conventional wheelbase trucks and wheel diameters can use standard 56-½-inch

[1,435-millimeter] track gauge in both tangent track and virtually all radius curves without regard to whether railroad or transit design standards are used for wheel gauge.

On an ideal light rail system, there would be no need for any variations of the track gauge, thereby producing a completely uniform environment for the wheel/rail interface. This may not be practical, particularly on systems that have tight radius curves and/or employ narrow flangeway groove rails. When mixed track gauges are employed, the designer should consider rail-grinding operations and the adjustment capabilities of state-of-the-art rail-grinding machines as a means of maintaining a reasonably consistent wheel-rail interface pattern.

The threshold radius at which it may be appropriate to alter the gauge in curved tracks will vary based on a number of factors related to the vehicles that operate over the track. Track gauge on moderately curved track can normally be set at the standard 56 ½ inches [1,435 millimeters] to accommodate common wheel gauges. As curves become sharper, more consideration should be given to ensure that sufficient freeplay is provided to prevent wheel set binding. Factors involved in this analysis are the radius of curve under consideration and wheel diameter, shape of the wheel flange, wheel gauge, and wheel set (axle) spacing on the light rail vehicle truck. Systems with mixed fleets and a variety of wheel and axle configurations must consider the ramifications associated with each and develop a compromise among the various requirements.

Conventional wisdom suggests that track gauge must be widened in curved track; however, this axiom is largely based on railroad experience with extremely large diameter wheels and very long wheelbases. By contrast, transit vehicles with smaller diameter wheels, short and narrow flanges, and short wheelbase trucks (i.e., axles are closer together) will often require no track gauge widening in curved track. Transit equipment may, therefore, require track gauge widening only on the most severely curved track segments and then only if the axle spacings, wheel flanges, and wheel diameters are large. Some equipment may need no track gauge widening at all, even at an 82-foot [25-meter] radius. As a guideline, it is recommended that systems that have numerous sharp curves select vehicles with shorter wheelbase trucks. Truck designs built with axles spaced 1800 to 1900 mm [about 71 to 75 inches] are generally satisfactory for universal use.

For trucks with wheel diameters less than 28 inches [711 millimeters] and axle spacing less than 74.80 inches [1900 millimeters], gauge increase will not be required even if AAR wheel flanges are used. Trucks with small diameter wheels and short axle spacings can also negotiate extremely small radius curves as low as 36 feet [11 meters] with only slight widening, usually about ¼ inch [5 mm]. Conversely, large diameter wheels, large flanges, and long wheelbases will require gauge widening at appreciably greater curve radii than smaller trucks. Trucks with large diameter wheels and a long wheelbase will generally have unsatisfactory operation on extremely sharp radius curves, are typically limited to curve radii of at least 82 feet [25 meters], and may require gauge widening on curves with radii less than 197 feet [60 meters]. If large, railroad-type wheel flanges are used in combination with narrow flangeway groove rails, even small track gauge increases are usually not possible because the gauge widening exacerbates the problem of back-to-back wheel binding.

Reduction, rather than widening of track gauge in curved track has been considered on several systems in Europe and by at least one agency in North America as a way to improve vehicle-

tracking performance when passing through reduced radius curves using groove rail. It is thought that reduction of track gauge could also reduce wheel squeal by limiting lateral wheel slip, which is believed to be a main source of such noise. See Chapter 9, Article 9.2.3 for additional discussion on this topic.

4.2.4 Curved Track Gauge Analysis

Requisite track gauge and flangeway dimensions in curved track must be determined analytically for each combination of vehicle truck factors. There are several graphical methods for analyzing this issue. The articles below will discuss two. The first, the Filkins-Wharton method, dates to the early 20th century. The second, a method known as the Nytram Plot, builds upon the Filkins-Wharton method.

4.2.4.1 Filkins-Wharton Flangeway Analysis

The tight wheel-to-track-gauge freeplay and small wheel flange profiles that were common on traditional street railways allowed for smaller flangeways than those needed for railroad service. Hence, girder rails that were rolled specifically for streetcar systems had narrower flangeways than the flangeways sometimes used by steam railroads. (Steam railroads often had embedded/paved track in urban warehouse and wharf districts and several designs of girder rails were once rolled specifically for that purpose.) The narrower flangeways of the girder rails designed for streetcar service were more conducive in areas with pedestrian traffic.

Mr. Victor Angerer was a Vice President of Wm. Wharton & Sons, a Philadelphia firm that was one of the leading special trackwork manufacturers of the early 20th century. In a paper presented before the Keystone Railway Club in 1913 and later reprinted in the *Electric Railway Handbook*,[1] Mr. Angerer said:

> *…theoretically for track laid to true gage every combination of radius of curve and wheel base of truck, with a given wheel flange, calls for a specific width of groove to make the inside of the flange of the inside wheel bear against the guard and keep the flange of the outside wheel from grinding against the gage-line and possibly mounting it. It is manifestly impracticable to provide guard rails with such a variety of grooves or to change the grooves of the rolled rail. The usual minimum of 1-9/16 inch is wide enough to pass the AREA standard flanges on a 6-foot wheel base down to about a 45-foot radius, and the maximum width of 1-11/16 inches down to about a 35-foot radius. On curves of larger radius the excess width should be compensated for by a corresponding widening of the gage. If the groove in the rolled rail is too narrow for given conditions, it must be widened by planing on the head side of the inside rail, to preserve the full thickness of the guard, and on the guard side of the outside rail to preserve the full head. Unusual wheel bases such as 8 feet or 9 feet may require widening of the gage on some curves. This widening of gage is necessary only to bring the guard into play when the groove is too wide for some one combination of wheel and flange. In T-rail curves the guard is formed of a rolled shaped guard, or a flat steel bar, bolted to the rail. In special work and curves in high T-rail track a girder guardrail is often used. This is desirable, as it gives the solid guard in one piece with the running rail. The idea that a separate guard can be renewed when it is worn out does not work out in practice, as it*

> *is usually the case that when the guard is worn the running rail is also worn to such an extent that it will soon have to come out also.*[1]

This excerpt provides still timely guidance in determining flangeway requirements, particularly for design of restraining rail systems, and evaluating the possible use of presently available groove rails.

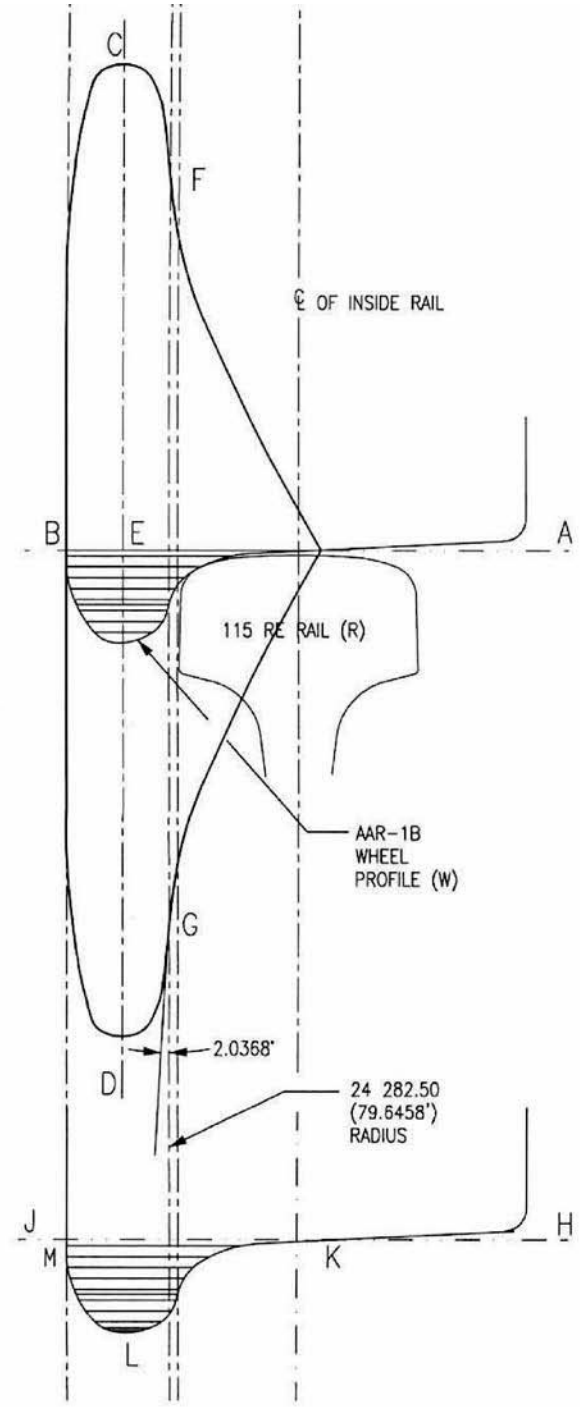
Around 1910, one of Mr. Angerer's employees, Mr. Claude W.L. Filkins, developed a graphical technique for determining the optimum track gauge and flangeway dimensions for any given conditions of truck dimensions, wheel diameters, and wheel profile. The Filkins-Wharton diagram analysis was a simple and effective technique to establish the minimum flangeway openings required to suit wheel flange profiles, curve radii, and axle spacings. The following describes the Filkins-Wharton diagram procedures.[1]

Figure 4.2.4 represents an AAR-1B wheel with a diameter of 28 inches [711 mm] placed on 115 RE rail on an 82-foot [25-meter] radius curve. In the illustration, the wheel is adjacent to the rail gauge line. On a conventional, rigid, non-steerable truck, the flange will never be sitting perpendicular to the curve radius but rather at a skew. That skew will vary in proportion to the wheelbase (distance between axles) of the truck, with longer wheelbases resulting in larger skews. In the example, the wheelbase is 72 inches [1828 millimeters]. Line A-B is the horizontal cut plane passing through the AAR-1B wheel profile [W] resting on the 115 RE rail head [R].

C-D-E represents a sectional view of the wheel at the plane defined by the top surfaces of the two rails. The line C-D-E is perpendicular to the axle. While the rail is actually curved, the length of rail head adjacent to section C-D-E is short enough to be considered a straight line.

The line F-G represents a perpendicular line to the radius line and forms an intersecting angle of 2.0368 degrees to the wheel axis C-D-E. For a static condition, all four wheels will produce an approximately similar angle for line F-G using the combination of curve radius and wheelbase. (In practice, this is not the case for a rolling truck because it will always be skewed to the track in the opposite direction from the curve.)

Geometric construction is applied to project the resulting flange profile on the plane H-J. Plane H-J is perpendicular to the rail head and radial to the curve. Projecting the points of the wheel in plan along the track arc to line H-J produces the outline K-L-M. Note that this shape is not the same as the wheel flange profile because the graphical exercise above is considering the entire space occupied by the flange below top of rail, including consideration of the angle by which the axles (and hence the wheels) are skewed to a radial line. In effect, the flange has been "fattened" to account for that skew. Outline K-L-M therefore represents the absolute minimum flangeway shape required to permit a vehicle truck with an AAR-1B wheel profile and the stated wheel diameter and wheelbase to negotiate the stated track radius. Track designers back in the early 20th century could then consult catalogs of available girder rails and select one which provided a flangeway at least that large. Naturally, additional flangeway clearance is still required to allow relatively free movement and to compensate for tolerances in the wheel mountings, wheel profiles, and track gauge tolerances—resulting in a wider actual flangeway. Flangeway depth must consider wheel tread wear and special trackwork design features as flange-bearing flangeways.



PARAMETERS:
- AAR-1B MODIFIED NARROW FLANGE WHEEL
- 82' (25 METER) TRACK CURVE
- 72" (1828) WHEEL BASE
- 28" (711) WHEEL DIAMETER
- 55 11/16" (1414.5) WHEEL GAUGE

Figure 4.2.4 Filkins-Wharton diagram for determining flangeway widths

Figure 4.2.5 illustrates the flangeway requirements using outline K-L-M considering both flangeways using 59R2 groove rail and standard track gauge and AAR wheel gauge. Note how the "fattened" wheel flanges just barely fit in the flangeway. Any appreciable amount of wheel flange wear would allow the wheelset to shift laterally, resulting in contact between the back of the wheel and the tram of the 59R2 groove rail on the opposite rail. That condition will be discussed further in Article 4.3 of this chapter.

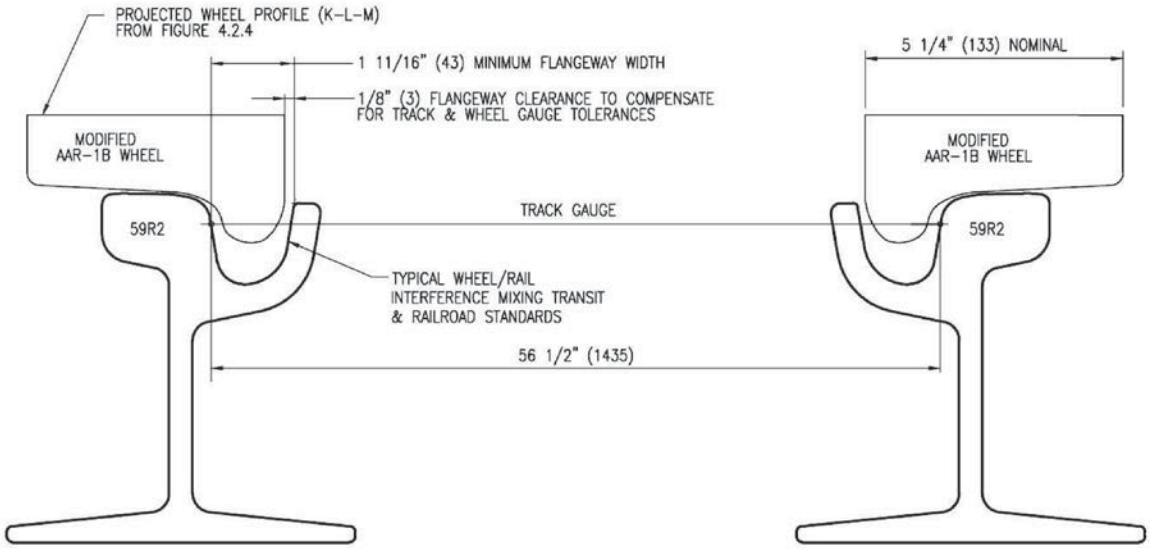


Figure 4.2.5 Filkins-Wharton plot to establish flangeways

See Chapter 5 for additional guidance concerning maximum flangeway width in embedded track and railway/highway crossings.

4.2.4.2 Nytram Plots—Truck-Axle-Wheel Positioning on Curved Track

Claude Filkins was limited to manual drafting methods and the accuracy of Filkins-Wharton diagrams was therefore limited when using drawing sheets of practical dimensions. Filkins-Wharton diagrams produced manually were forced to graphically shrink track gauge and wheelbase in order to depict an entire truck assembly on a reasonably sized drafting sheet. The method also does not consider dynamic truck behavior, but presumes the truck is always square to the track.

A modified version of the Filkins-Wharton diagram, referred to herein as the Nytram plot, has therefore been developed taking advantage of the power of computer-aided design and drafting (CADD) as an analytical tool. CADD provides the track designer with the ability to develop a full-sized picture of the entire vehicle truck positioned on a curved track, including rotation of the truck to mimic actual behavior. These CADD images can then either be plotted at reduced scale, or selected portions of the diagram can be printed at full size.

To illustrate the methods involved, a series of figures have been developed that illustrate the fundamentals of adapting track gauge to wheel gauge, wheel contour, and positioning of a truck on a segment of curved track. The figures consider the following parameters:

Wheel Profile	Modified AAR-1B 5 ¼ inches [133 millimeters] overall width
Wheel Diameter	28 inch [711 millimeters]
Wheel Gauge	55.6875 inches [1414.5 millimeters] (AAR standard)
Wheel Back to Back	53 $^{3}/_{8}$ inches [1356 mm] (AAR standard)
Axle Spacings	74.80 inches [1900 millimeters]
Curve Radii	82 feet [25 m], 300 feet [91.4 m] and 600 feet [182.9 m]

An AAR wheel profile and gauge has been used in the examples so that the variables are limited to curve radius. Projects that wish to use groove rails with narrow flangeways need to consider transit profile wheels with narrow flanges and wider back-to-back wheel gauge.

4.2.4.2.1 Nytram Plot—Wheel Profile Sections

The first step in developing a Nytram plot is to take sections of the wheel at several elevations at, above, and below top of rail. **Figures 4.2.6** and **4.2.7** show horizontal sections of a selected wheel profile that have been derived at the gauge line elevation, at the top of rail, and, where appropriate, at the top of a restraining rail positioned $^{3}/_{4}$ inches [19 millimeters] above the top of the running rails. If a restraining rail is present at a different elevation, a different section would obviously be required. Note how the length of each section (parallel to the rail) is dependent on the diameter of the wheel. Large diameter wheels will have longer wheel sections and will occupy more space in the flangeway, especially in curves. LRT systems with mixed vehicle fleets with wheels of varying diameters will need to consider each wheel separately.

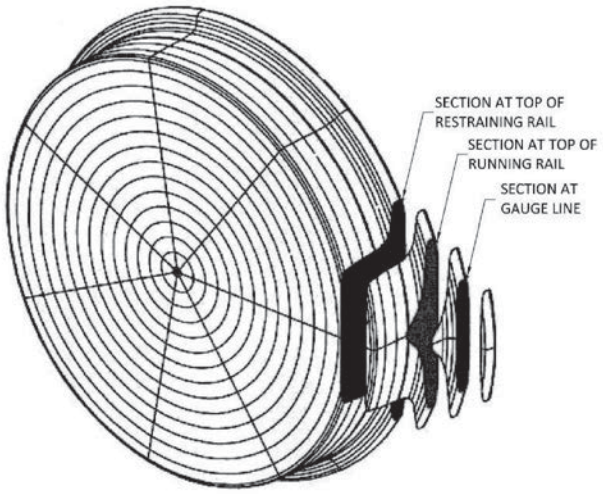


Figure 4.2.6 Wheel sections for Nytram plot—oblique view

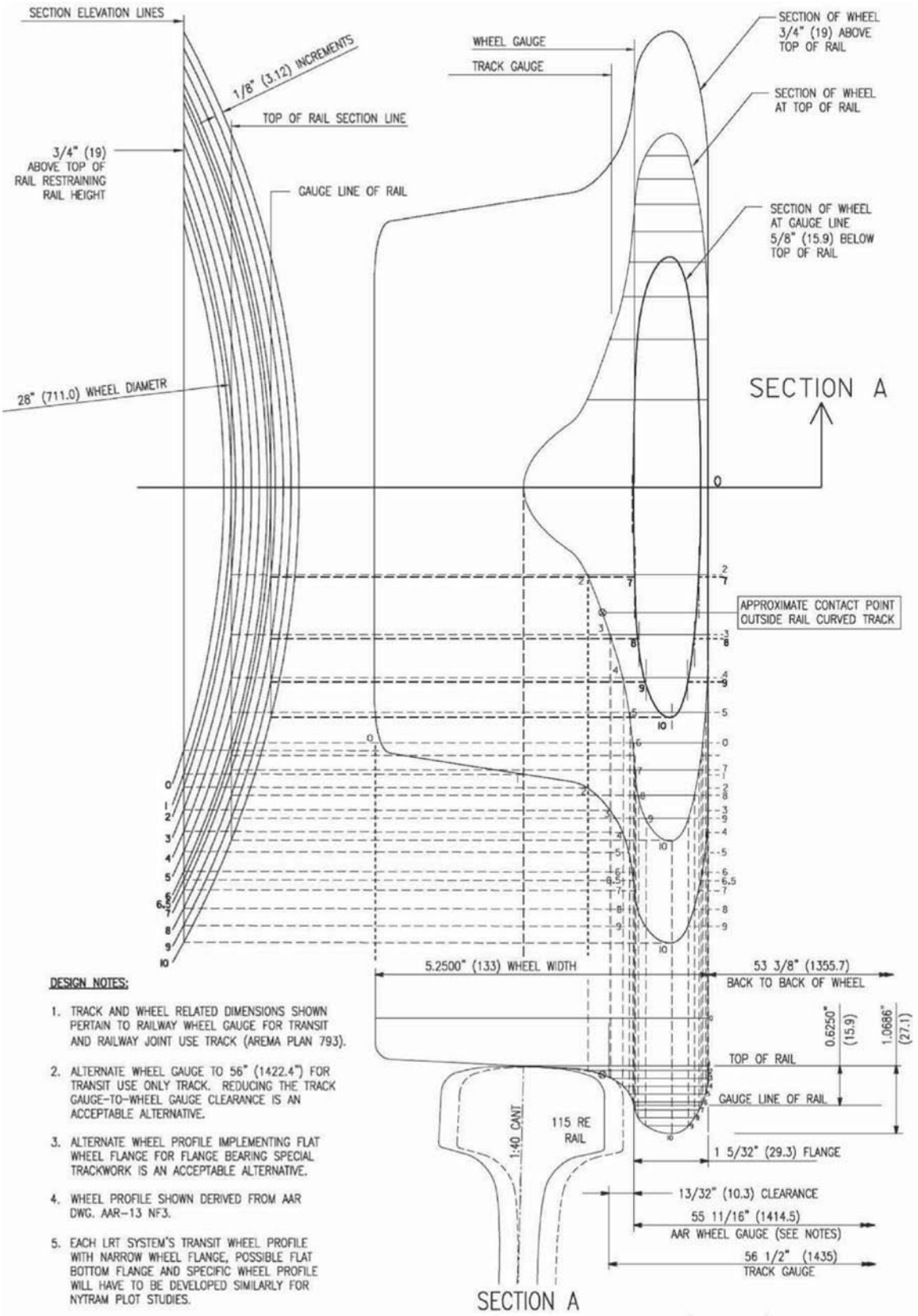


Figure 4.2.7 Wheel sections for Nytram plot—modified AAR-1B transit wheel

Figure 4.2.7 illustrates the details of the process. Identification points are established on the surface of the wheel flange to define points of horizontal flange sections and assigned numbers from zero up to 10. Those points also define circular arcs in $^{1}/_{8}$-inch [3.175-mm] increments along the wheel surface, as seen in the elevation view of the wheel on the left side of the figure. Projecting points 0 to 9 from both sections as shown, a horizontal section or "footprint" of the wheel can be developed at various heights above or below the top-of-rail elevation.

Using these wheel sections, the actual positions of the vehicle truck axles and wheels can be superimposed on a section of curved track of any specific radius so as to simulate the complete truck in a skewed position. This allows the designer to determine the maximum "angle of attack" of the leading wheel with the outer rail, the points of wheel flange contact with both running rails and the restraining rail (if present), and the wheel flange-to-rail clearances. It will also determine whether any wheel binding will be present should the track gauge be too tight.

4.2.4.2.2 Nytram Plots—Static Condition

The next step is to graphically "assemble" a complete vehicle truck by mounting the wheel profile sections on imaginary axles and positioning those axles the correct distance from each other. That assembly is then positioned on a graphical representation of the track drawn to scale at the curve radius of interest, perpendicular to the radius line and with the flanges all equidistant from the two rails.

Figure 4.2.8 illustrates a stationary transit vehicle truck with a 28-inch [711-mm] diameter wheel, AAR wheel gauge, and an axle spacing of 74.80 inches [1900 mm] positioned on an 82-foot [25-meter] radius curve. This figure was developed by following these steps:

- Develop three curve centerlines using radii of 82, 300, and 600 feet [25, 91.44, and 182.88 meters, respectively]. (Figure 4.2.8 is actually drawn as an 82-foot/25 meter radius curve so as to more clearly illustrate the conditions. The calculated dimensions for the other radii have been added to the graphic for comparison purposes.)
- Develop the track gauge lines concentric with the track centerline. In this exercise, standard 56.5-inch [1435-millimeter] track gauge has been used.
- Develop the vehicle truck centerline perpendicular to the track radius line, measuring half the axle spacing in each direction, and placing the center of each axle on the centerline of track.
- Develop the truck axles perpendicular to the centerline of the truck.
- Place the vehicle wheel sections developed in Figure 4.2.7 on the axles spaced at the back-to-back distance perpendicular to the axle centerline. The truck should now be centered on and square to the track.
- To establish wheel flange clearances to the gauge line of the track, graphically measure the distances from the gauge line of the rail to the closest point on the wheel profile outline at gauge line elevation.

These measured dimensions are normal to the rail but not parallel to the axles. Note that, because of the skew of the truck, these clearances will always be less than the wheel-gauge-to-track-gauge freeplay that will exist on tangent track. Note also that these clearance dimensions

vary depending on whether the measurement is done at gauge line elevation or at some other elevation at or below the top-of-rail plane. When considering modern rail sections with compound radius gauge corners paired with a conformal wheel profile, a precise evaluation will typically reveal that the first point of contact between wheel and rail occurs at a point about $^{3}/_{8}$-inch [9.5-mm] beyond the gauge line and not at the gauge face of the rail. However, this distinction can generally be neglected for ordinary analysis.

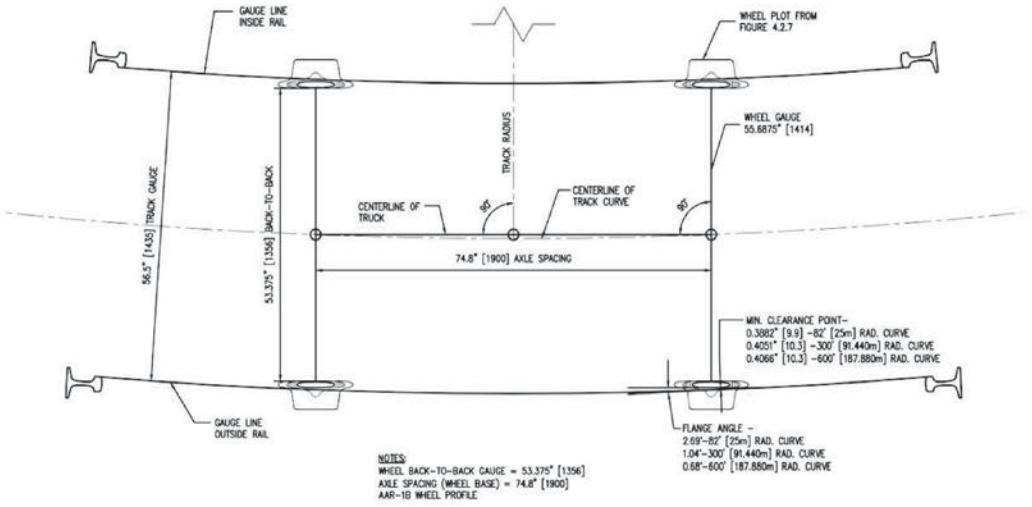


Figure 4.2.8 Static Nytram plot

Similar plots (not shown here) were undertaken with the same truck parameters for curves with 300-foot [91.44-meter] and 600-foot [182.88-meter] radii, and the clearance results were added to Figure 4.2.8. The intersection angles between the perpendicular truck and the tangent point to the track arc have been determined graphically and are shown for the three curve radii for comparison.

The above Nytram description and illustration depicts a static truck superimposed on a curve perpendicular to the radius line so as to illustrate the basic concepts. To determine the operational flangeway widths and the angle of attack between the wheels and the rails, the actual dynamic truck skewing must be considered, as described below.

4.2.4.2.3 Nytram Plots—Dynamic Condition

As a next step, so as to simulate the steering action of the vehicle truck traversing through the various curves, a set of drawings with the same truck parameters as above has been developed. These next figures simulate the typical steering action that occurs when a truck leaves tangent track and enters a curve. The leading outside wheel on the truck encounters the curved outside rail resulting in steering or deflecting of the lead axle and the truck.

Figure 4.2.9 illustrates the same vehicle truck as shown in Figure 4.2.8. The truck has been rotated about the center of the truck (Point "A") in a direction opposite that of the curve in the track until a first point of contact is found between a wheel and a rail. For track without restraining rail, this first point of contact will always be at the wheel on the leading axle that is riding on the outer rail of the curve, here designated as Wheel "B." This mimics the condition that occurs when a truck first enters a curve from a segment of tangent track.

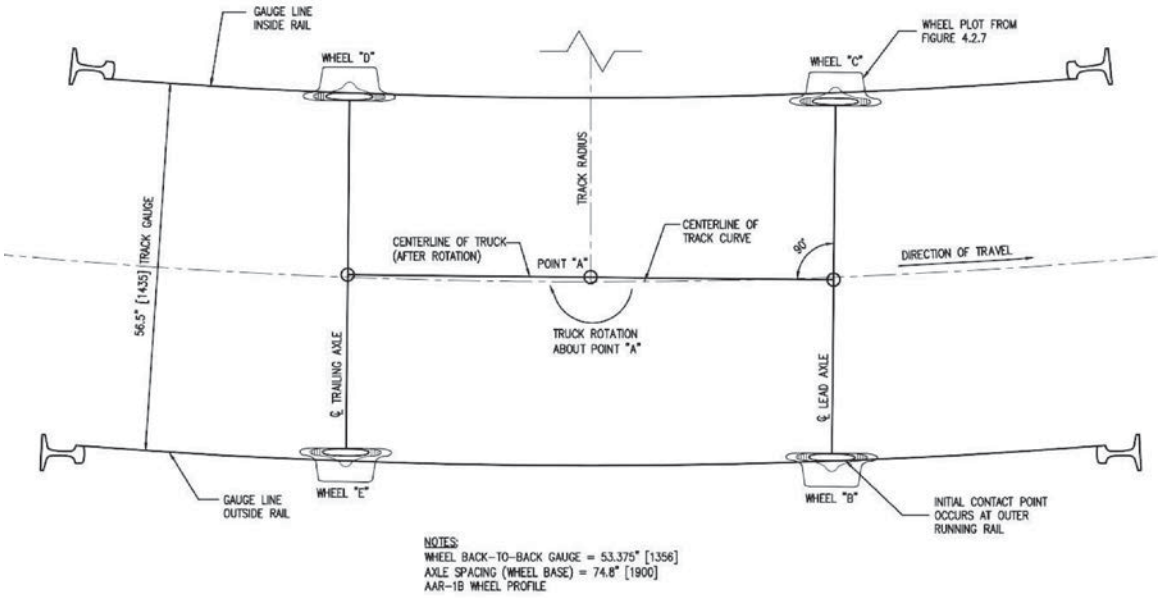


Figure 4.2.9 Nytram plot—rotated to first point of contact

As a point of order, it should be noted that the truck does not actually instantaneously rotate about Point "A" at the beginning of a curve. The rotation shown in Figure 4.2.9 is merely a graphical tool for approximating the net effect of a series of events. Those events include an initial wheel contact as the truck continues to roll straight for a short distance into the curve until initial flange contact is realized. Concurrently, the effects of differential rolling radius on conical wheel treads will be felt due to the shorter rolling distance along the inner rail. In combination, these events have the same net effect as the graphical truck rotation.

Figure 4.2.10 illustrates the next step. Once the leading outside wheel initially contacts the outer rail, the rolling wheel along the inner rail (which has a shorter distance to travel) causes the truck to continue to rotate, seeking a second wheel-flange-to-rail contact point. However, this additional rotation will not occur about Point "A," but rather about that first point of contact at Wheel "B," as identified in Figure 4.2.9. Typically, the second point of contact occurs at the inner wheel of the trailing axle (Wheel "D"); however, trucks with moderate self-steering capability may not encounter the second contact point.

With the truck in this fully rotated condition, it is then possible to graphically measure various parameters including

- The angle of attack of the lead wheel to the outside running rail.
- The wheel-flange-to-rail clearances at each wheel.

- The absolute minimum flangeway widths necessary to permit free passage of the flanges.

The last bullet point above becomes an important issue in embedded track using tee rail because if the flangeways are too narrow, unintentional contact could occur between the backs of the wheels and the paving material that defines the edge of the flangeway. It is necessary to add a factor to that dimension to account for construction/fabrication tolerances and also to allow some running clearance.

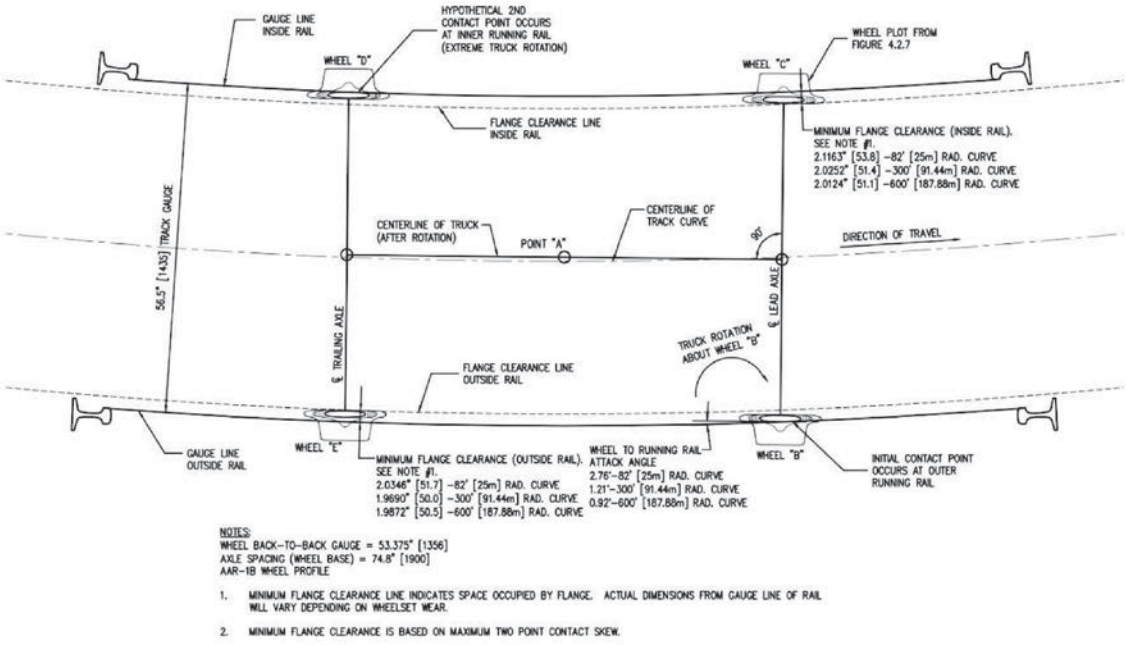


Figure 4.2.10 Nytram plot—rotated to second point of contact

It is important to note that the analysis above is considering an idealized condition where both the wheels and rails are new and unworn and the track gauge has been constructed with a zero tolerance. Worn wheels and rails and wide track gauge will result in larger angles of attack and larger values of wheel-flange-to-rail clearance. Because of these issues, wider flangeways than the dimensions determined will always be required in plain, non-guarded track.

This type of interface study should be undertaken jointly by the project's vehicle and track designers. Incorporation of factors to account for peculiarities of the truck design, as identified by the vehicle engineers, may be appropriate. For example, the Nytram drawings presume that the truck remains absolutely rectilinear and do not account for either potential axle swivel that might be permitted by a flexible primary suspension system at the journals or any possible twisting or racking of the vehicle truck into a parallelogram configuration. These conditions may vary in each manufacturer's truck design.

4.2.4.2.4 Nytram Plots Considering Restraining Rail

The drawings as developed above do not consider restraining rail; however, a measured inside rail flangeway width has been stated on the drawings as a reference. If the use of restraining rail

is selected on a system due to restricted sharp radius curves, then a similar scenario should be undertaken using the parameters of the vehicle truck and track system to establish the flangeway.

Figure 4.2.11 illustrates the truck shown in Figures 4.2.8 through 4.2.10 statically mounted on a curve with a restraining rail mounted along the inside rail of the curve. For purposes of this trial, the restraining rail has been positioned flush with the top of the running rails, and the flangeway width has been set at 2 inches [51 mm].

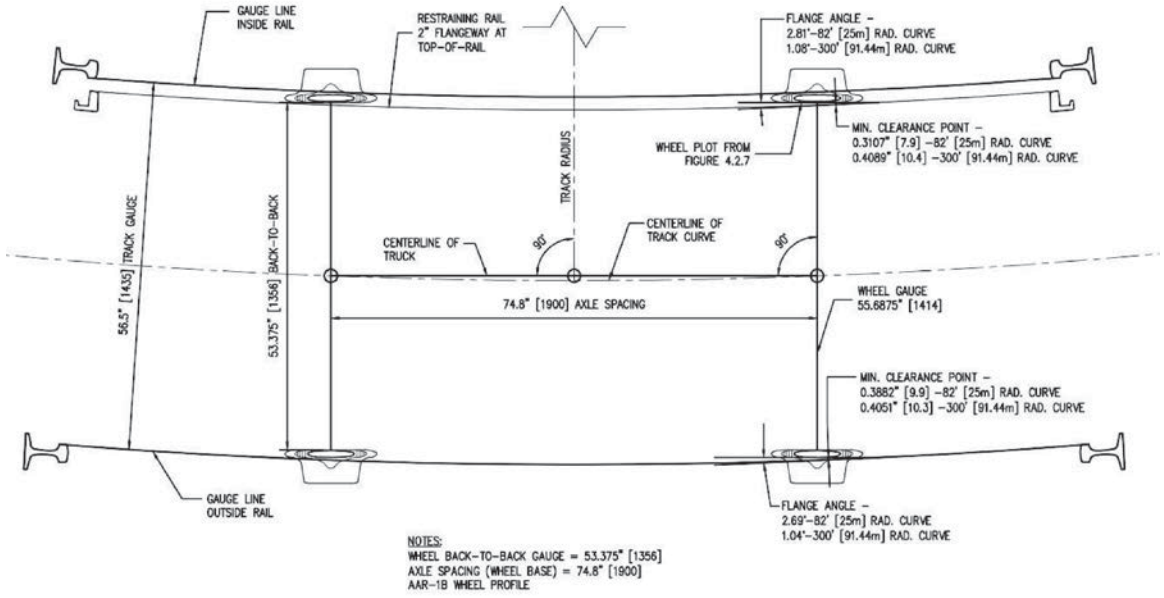


Figure 4.2.11 Static Nytram plot with restraining rail

Figure 4.2.12 shows the truck rotated to the first point of contact, which now occurs not at Wheel "B" but rather between the restraining rail and the back face of Wheel "C." In **Figure 4.2.13**, the truck is rotated about that first point of contact at Wheel "C" to find the second contact point. For curves that do not have a restraining rail on the outer rail, that second point of contact will still occur at Wheel "D." Clearances and angles can then be measured graphically as previously discussed.

Note how, just as the calculated clearances will vary depending on track gauge, the width of the flangeway is critical as well. If the flangeway is wide, the first point of contact may still occur at Wheel "B." In such cases, the restraining rail may not come into play until a combination of wheel flange wear and rail gauge face wear results in some vehicle trucks contacting the restraining rail at Wheel "C."

For extremely sharp radius curves using double restraining rails, the same procedures are required to establish both flangeway widths.

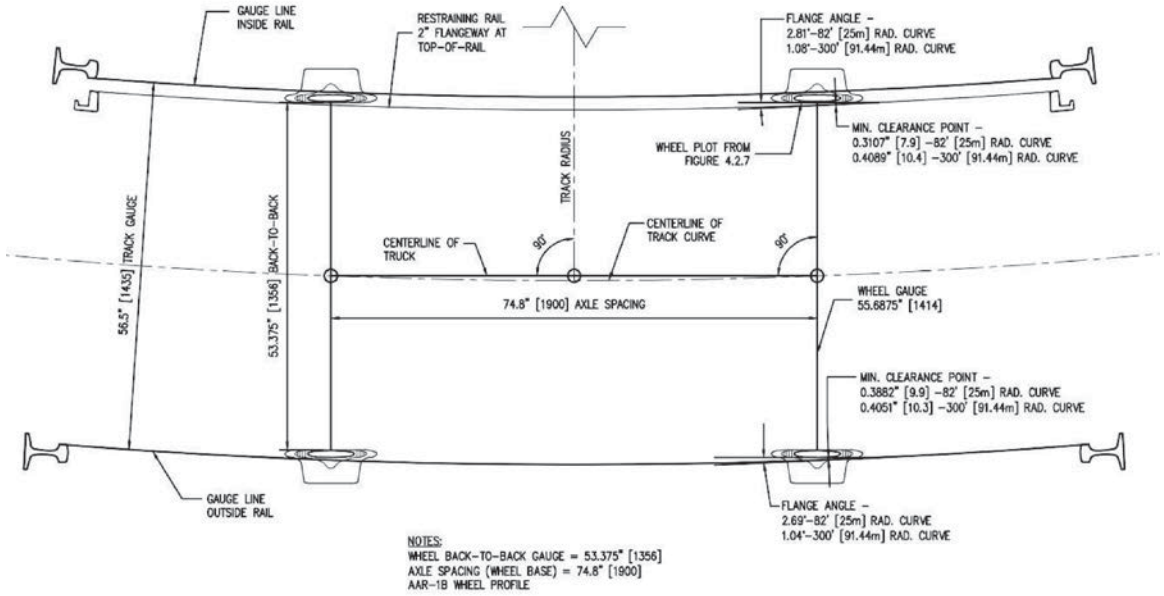


Figure 4.2.12 Nytram plot with restraining rail—rotated to first point of contact

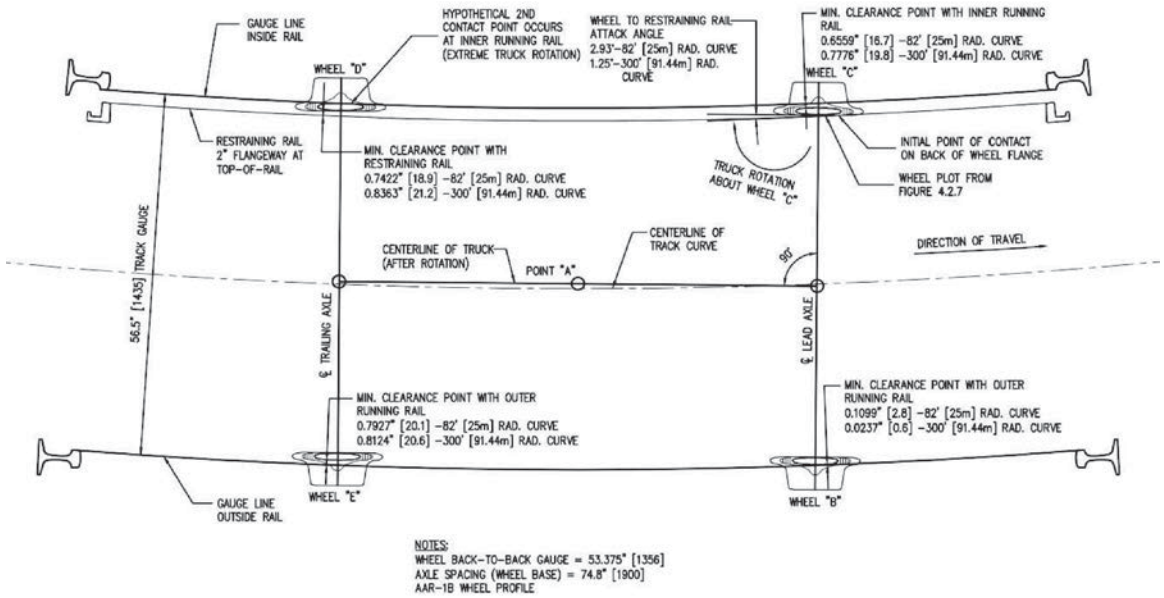


Figure 4.2.13 Nytram plot with restraining rail—rotated to second point of contact

All of the illustrations above use AAR wheel profiles and wheel gauge. If the same analysis is performed using a transit wheel profile and/or wheel gauge, different values will ensue. In general, the use of AAR parameters, particularly their wheel mounting gauge, requires a wider flangeway than when using transit wheel parameters.

As a guideline, it is recommended that the inside restraining rail flangeway width be set to provide shared contact so that the inside back face of the wheel makes contact with the restraining rail face while the outside wheel is simultaneously contacting the gauge corner of the outside rail. This will theoretically divide the lateral steering force between both wheels and rails. However, the following conditions must be recognized:

- The ability to precisely achieve dual contact must be tempered by the practical fabrication dimensions and tolerances. It is impractical to specify flangeway widths to a fabrication tolerance finer than +/- $^{1}/_{16}$ inch [1.6 millimeters].
- In practice, simultaneous dual contact may not occur immediately; however, wear at either the gauge face of the outside running rail or the working face of the restraining rail will eventually lead to routine shared contact.
- On any LRT system of appreciable size, variations in wheel wear on various cars plus variations in construction and maintenance tolerances of the track at any given location will guarantee that no two cars will track through any particular curve exactly the same way. Some vehicles will end up always being steered solely by the high rail. Other vehicles might have 100% of their steering via the low restraining rail. Some vehicles may result in lateral load sharing, but it will rarely be a 50-50 split and will likely fluctuate with variations in the dynamic track gauge because of tolerances and railhead deflection.
- Some small amount of lateral load will be transferred via top-of-rail friction; however, that will be erratic since wheel slip—both lateral and longitudinal—is essential during negotiation of tight radius curves. It is not a perfect system where loadings can consistently be predicted with mathematical and mechanical precision.

In spite of some shortcomings, the Nytram plot concept described above has been used on many projects with appreciable success. By careful trial analysis, varying the parameters, an optimum configuration can be derived. In general, keeping the truck as close as possible to being square to the track will result in the optimal long-term performance.

See Article 4.3 of this chapter for a discussion of restraining rail, including pros and cons regarding its use. Even if no restraining rails are used, the Nytram plot is a useful tool for identifying the minimum flangeways necessary in curved embedded track. On more than one occasion, the flangeway formed in the pavement next to sharply curved, embedded tee rails has been discovered to be too narrow when the back sides of the wheels began grinding into the concrete.

4.2.5 Rail Cant and Wheel Taper—Implications for Track Gauge

Rail cant is a significant factor in wheel-to-rail interface. Cant describes the rotation of the rail head toward the track centerline. It is intended to complement conical wheel treads in promoting

self-steering of wheel sets through curves. The cant also moves the vertical wheel loading away from the gauge corner of the rail and toward the center of the ball of the rail head. Tee rails are generally installed at 1:40 cant in both tangent and curved track. Additional rail cant could be considered at short radii embedded curves below 300 feet [91 meters] at the low rail. This additional cant can be applied by installation of a 1:30 cant shim at the time of construction. The additional cant design duplicates the asymmetrical offset rail head grinding of the low rail, retaining the true crown profile of the rail head. This procedure has been used to reduce wheel squeal on at least one LRT system with favorable results.

Zero cant is usually specified through special trackwork so as to simplify the design and fabrication of trackwork components. Canted special trackwork is now often specified for high-speed rail operations, but there is little benefit for doing this at the relatively low speeds commonly reached on LRT operations.

When using tee rail, rail cant is achieved by using one of the following:

- Concrete cross ties with the rail cant cast into the rail seats.
- Canted tie plates on timber cross ties.
- Canted direct fixation rail fasteners on a flat concrete invert.
- Flat direct fixation rail fasteners on a canted concrete invert.

In embedded track, the rail cant can be incorporated into the gauge ties that are usually used to hold the rails. Modern groove rails such as 59R2, which effectively incorporate cant into the rolled head and can therefore be laid on flat fasteners, are preferable to the older designs (such as 59R1), which must be placed on canted fasteners if cant is desired.

4.2.5.1 Tapered Wheel Tread Rationale

Both railroad and the majority of transit wheel tread designs are typically tapered to be shaped like a truncated cone. A cone that is lying on a flat surface will not roll in a straight line. But a pair of conical wheels that are rigidly mounted on a solid axle, each supported on a single edge—such as at each rail—can be made to follow a straight path provided the axle axis is held rigidly at right angles to the direction of travel. Railway wheel design takes advantage of this geometric relationship to facilitate self-steering of trucks through gentle curves without requiring interaction between the gauge side of the high rail head and the wheel flanges.

The usual conicity of the wheel tread is a ratio of 1:20. This results in a wheel that has an appreciably greater circumference close to the flange than it has on the outer edge of the wheel tread. In curved track, this differential moderately compensates for the fact that the outer rail of a curve is longer than the inner rail over the same central angle. The wheel flange on the outer wheel of the leading axle of a conventional solid axle truck shifts toward the outer rail when negotiating a curve; hence, that wheel rolls on a larger circumference. Meanwhile, the inner wheel flange shifts away from that rail and that wheel rolls on a smaller circumference. Thus, the outer wheel will travel forward a greater distance than the wheel on the inner rail even though they are both rigidly attached to a common axle and hence have the same angular velocity. As a

result, the axle assembly steers itself around the curve just as a cone rolls in a circle on a table top.

Note that rolling radius differential is maximized when the wheel and axle set is free to shift laterally an appreciable amount. An actual cone has a fixed slope ratio; hence, it can smoothly follow only one horizontal radius. A wheel and axle set with tapered wheels, on the other hand, can assume the form of a cone with a variable side slope by shifting the freeplay left and right between the wheel flanges and the rails. Hence, larger values of track gauge to wheel-gauge freeplay can be beneficial in that regard. However, that larger freeplay also allows significant truck skewing and increases the angle of attack between the leading wheel and the outer rail of sharp curves.

Railroad wheel sets mounted at AAR standard wheel gauge and tapered at 1:20 theoretically eliminate flanging on curves with radii over 1900 feet [580 meters], which is about a 3-degree curve. Many railroad design criteria specify 3 degrees as the desirable maximum curvature. Below that radius, contact between the outside wheel flange throat and the gauge corner of the outside rail provides a portion of the steering action. Nevertheless, tapered wheels still provide a significant degree of truck self-steering that reduces flanging on curves with radii as small as 328 feet [100 meters]. For sharper curves, flanging is the primary steering mechanism.

However, wheel sets that have reduced freeplay between wheel gauge and track gauge will commence flanging at a higher curve radius than a wheel set using AAR wheel gauge. Therefore, transit wheels self-steer only on relatively large radii curves, due to the fact that the reduced freeplay between wheel gauge and track gauge allows only very limited differential rolling radii on a conical wheel before the wheel begins flange throat contact with the gauge corner of the rail.

Wheel profiles that have a cylindrical tread surface do not self-steer through curves of any radius; hence, flanging is the primary steering mechanism. Conical wheels that are not re-trued regularly also lose their steering characteristics because the contact patch becomes excessively wide as a significant portion of the wheel tread matches the contour of the rail head. Hollow worn wheels develop a “false flange” on the outer portion of the tread and can actually attempt to steer the wrong way as the rolling radius on the tip of the false flange can be equal to or greater than the rolling radius on the flange-to-tread fillet. The importance of a regular wheel truing program cannot be overstated, and track designers should insist that vehicle maintenance manuals require wheel truing on a frequent basis.

The center trucks on 70% low-floor light rail vehicles do not have wheels rigidly mounted on a solid rotating axle. Instead, as described in Chapter 2, the center truck design consists of low-level “crank axles” providing independently rotating wheels (IRWs) mounted on stub axles. Since these pairs of wheels are not forced to have the same rotational velocity, these trucks derive no self-centering benefit from tapered wheels. They also behave differently in curves, and the steps described for Nytram plot truck rotation in Article 4.2.4 may not apply. For additional insight into low-floor car performance and design refer to *TCRP Report 114: Center Truck Performance on Low-Floor Light Rail Vehicles*.

4.2.5.2 Rail Grinding

Rail grinding is essential in transit track maintenance and is discussed in Chapters 9 and 14 of this Handbook. However rail grinding can also play a role in new track design.

Normal rail grinding developed around the needs of the freight and passenger railroad industries and is focused on removal of rail head defects with the objective of extending the service life of the rail. The usual rail-grinding operation on freight railroads involves

- Removal of rail head corrugations.
- Removal of top-of-rail and gauge corner defects, such as rolling contact fatigue checking, and provision of gauge corner relief to defer re-initiation of defects in both the top-of-rail and in the gauge corner.
- Reshaping the top-of-head to a preferred rail head contour.

The rail-grinding service industry has developed equipment and detailed procedures tailored to the needs of its railroad customers. However, the needs of a rail transit system are distinctly different than those of a railroad. Differences include the following:

- Freight railroads need to have very long stretches of rail ground during relatively short work windows. Work windows on transit operations can be even shorter.
- Freight railroad rail grinding can use relatively coarse grinding stones since the heavy wheel loads of freight equipment will quickly erase the “signature” grinding pattern or marks. However, the comparatively small wheel loads of rail transit can take a very long time to erase the signature grinding marks. In the meantime, the wheel/rail interface oscillations initiated by the coarse grinding marks can grow into new problems, including high-pitched noise and even new rail corrugations.
- Rolling contact fatigue type rail defects, which are common under railroad loadings, generally do not occur under transit loadings because transit’s light wheel loads do not stress the rail steel anywhere near as much as freight loads.
- Rail corrugation patterns in rail transit are appreciably different than those in freight track.
- Transit systems typically need an initial rail grinding to remove mill scale and light rust so that signal circuits will shunt reliably. This is not a concern for freight railroads since the heavy axle loads will quickly wear away any such surface contamination.

Noise that originates at the wheel/rail interface has always plagued rail transit systems, and the condition of the rail head surface is a major contributor to noise. Rail grinding is utilized to remove two principal categories of unwanted surface imperfections that are a source of the noise. These are

- “Mill scale,” both from the original manufacturing rolling of the rail and subsequent heat treating processes.
- Rail head corrugation formed during operation, which has proven to be a detriment and key source of noise.

However, conventional rail-grinding practice in North America has evolved around the needs of the freight railroads, who generally have different concerns relative to rail imperfections and the

general quality of the finished grinding. For example, mill scale is completely a non-issue for freight railroads. Finish tolerances are also much less, in part because the high axle loads of freight equipment will quickly roll smooth any coarseness from the grinding stones. But these matters are very important when grinding transit rail since the light axle loads will not smooth out any discontinuities.

Conventional freight railroad grinding methods produce two undesirable conditions:

- Transverse rail-grinding signature score patterns gouged into the rail head.
- A series of flat grinding facets across the rail head, each approximately ½ to $^{7}/_{8}$ inch [13 to 19 millimeters] wide.

Both of these conditions lead to wheel/rail noise. The latter can, under light rail transit loadings, result in an erratic longitudinal tracking pattern by the wheels. If rail transit grinding is not carefully controlled with respect to grinder pass speeds and number and size of cross head facets, the resulting rough conditions could result in significant wheel/rail noise. Transit rail grinding therefore must be far more carefully controlled than freight railroad grinding. The types of stones used, the grinding pass speed, and the width of the facets must all be carefully controlled.

"Acoustic grinding," providing a clean duplication of the original rail head profile without signature grinding marks and flat facets is therefore much preferred for transit service. It involves both finer grit grinding stones and additional passes so as to virtually eliminate the facets and more precisely achieve the desired rail head profile. With effort, it is possible to achieve a rail head surface finish within 0.5 mil [about 13 microns] of the theoretical rail head contour. For further discussion on rail grinding requirements and methodologies refer to Chapters 9 and 14.

4.2.5.3 Asymmetrical Rail Grinding

The objective of rail grinding on railroads is usually to remove rail surface imperfections such as corrugation and rolling contact fatigue (RCF) defects. A relatively recent practice (since about 1990) has been rail grinding designed to alter the location of the wheel/rail contact band. By grinding an asymmetrical profile on the rail head and having distinctly different contact band locations along the high and low rails of a given curve, the location of the contact patch on the tapered wheel tread can be optimized, thereby changing the rolling radius of wheels on a common, rigid, fixed wheel/axle assembly. Given a specific wheel contour, a special grinding pattern can be created for each curve radius, thereby optimizing the ability of the leading axle of a truck to steer through that curve.

However, on curves sharper than the self-steering radius, this benefit cannot be fully realized by the trailing axle since it will always follow a slightly different path than the leading axle. Asymmetrical grinding also cannot assist curving of trucks with stub axles and independent rotating wheels.

4.2.5.4 Variation of Rail Cant as a Tool for Enhancing Truck Steering

Rail cant variation, as stated previously, can improve the rolling radius differential on standard rail head profiles in a manner similar to that achieved by asymmetrical rail grinding. Aside from the structural implications of loading the rail closer to or further from its vertical axis, greater or lesser amounts of cant can be beneficial by altering the location point on the tapered wheel tread that

contacts the rail. Installing rails with no cant creates a contact zone or wear strip that is close to the gauge corner of the rail. In rails installed with 1:40 or 1:20 cant, the contact patch progressively moves further away from the gauge corner of the rail. Note that the greater the rail cant (e.g., the smaller the second figure in the cant ratio), the smaller the rolling radius of a tapered wheel, which increases the self-steering effect when wheels shift to maximum off-center position.

Figure 4.2.14 illustrates the theoretical contact patch locations measured from the vertical centerline of 115 RE rail with an 8-inch [203.2-mm] crown radius. The lateral distance between the contact patches for 1:40 and 1:20 cants is 0.20 inches [5.1 millimeters]. This shift results in a decrease in wheel circumference at the contact point of 0.062 inches [1.6 millimeters] for a wheel with a 1:20 taper. While this may appear to be insignificant, if the higher cant is applied to the inside rail, it will increase the amount of curvature the wheel set can negotiate without flanging by a significant amount. For example, a light rail wheel set at transit wheel gauge will flange at about a 4,000-foot [1220-meter] radius if both rails are at 1:40 cant. But, if the low rail is canted at 1:20 while the high rail remains at 1:40, then the threshold radius for flanging could drop to as low as about 2,500 feet [750 meters]. Note also that the difference between the center of the rail and the center of the contact patch will vary with the crown radius of the rail. Wheels running on rails with smaller crown radii, such as the 8-inch [203.2-mm] crown radius that AREMA introduced to 115 RE rail in 2009, will behave slightly differently from rails with flatter heads.

Cant differential, in effect, mimics asymmetrical rail profile grinding. However, the application of increased cant at the low rail in curved track can be considered even if asymmetrical rail grinding is practiced.

Construction issues that ensue from a decision to use differential cant include the following:

- In ballasted tracks, any curves with non-standard cant will need to employ different concrete ties (or different tie plates on timber ties) than for tangent track. Further, the curve ties would have right- and left-hand orientations that would have to be carefully monitored during track construction. There would also be inventory issues associated with having several designs of cross ties (or tie plates) that probably will not look all that much different at first glance.
- In direct fixation track, the different rail cant could be achieved when pouring the plinths or by placing tapered shims beneath the rail fasteners. Jigs for top-down construction that facilitate adjustments to the rail cant are available. Either approach would be vastly preferable to having several different types of rail fastener in the track system, particularly if the differences between the fasteners are not visually obvious at first glance. Simplification of maintenance inventory is greatly appreciated by maintainers and provides better assurance that the right product will be used at the right location.
- Differential cant is relatively easy to achieve in embedded track. Either the ties can be fabricated with the ends canted, or tapered shims can be inserted between the ties and the base of rail. However, in the case of tracks built with groove rails (many of which incorporate normal cant into the head by design), actually inclining the rail will, in effect, lower the lip of the tram with respect to a plane defined by the tops of the two running rails. If the track design depends on the tram to act as a restraining rail, the tram will be less effective because it sits lower.

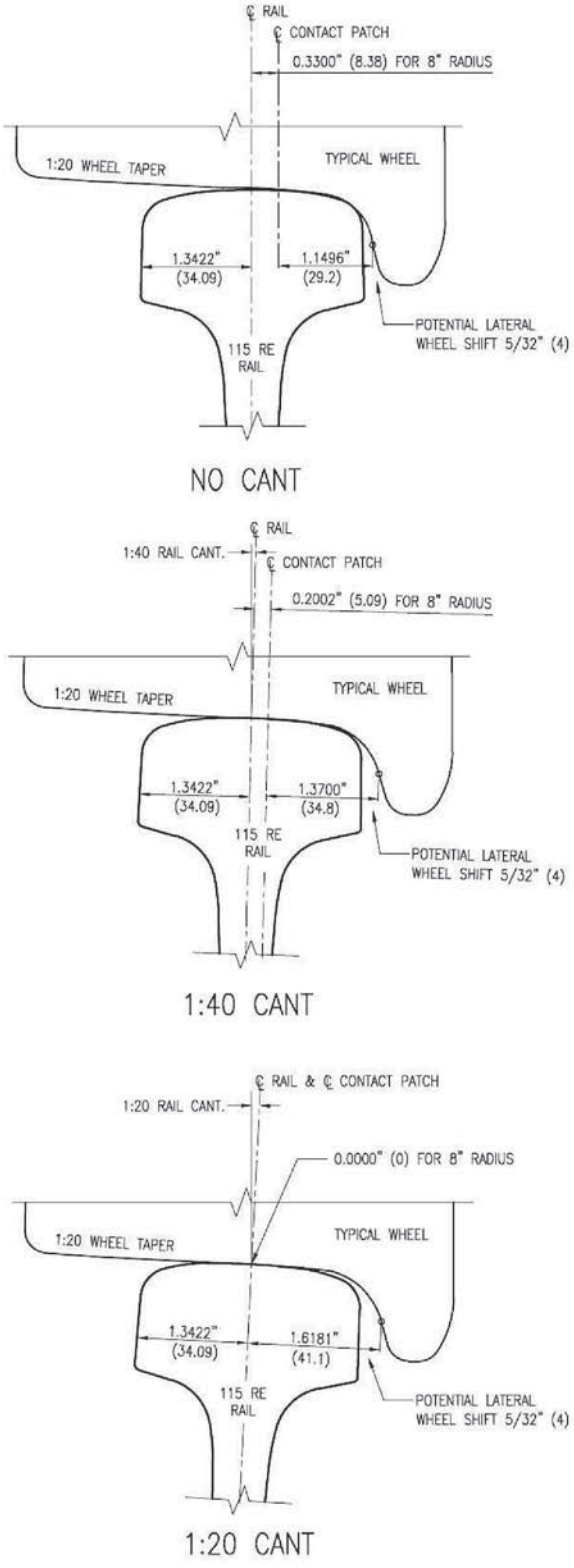


Figure 4.2.14 Rail cant design and wheel contact

The benefits of differential cant, like those of asymmetrical rail grinding, decline as the wheels and rail wear. As wheel treads wear toward a flat or hollow profile and rails wear to conform with the wheel profile, self-steering capabilities decline. Moreover, once the rail has worn, the contact patch will need to be restored to its as-designed location by asymmetrical rail profile grinding. Records must therefore carefully designate where zones of either asymmetrical grinding or differential cant exist so that future maintenance grinding operations can make adjustments to the angles of the grinding stones.

The true benefit of using a tapered shim versus asymmetrical grinding during initial construction is the retention of the original rail head profile and specifically the crown radius. The desire in rail and wheel maintenance is to retain or reestablish the original rail profile contour and restore the designed wheel profile by precision rail grinding and wheel truing, respectively.

4.2.6 Construction and Maintenance Tolerances—Implications for Track Gauge

The most precisely calculated standards for track gauge and flangeways will be of no value if the track is not constructed and maintained in a manner that ensures that the design intent is achieved in practice. Obviously, perfectly constructed and maintained tracks are not possible, and the cost of achieving such perfection would probably exceed the value of the benefits that would ensue. Accordingly, tolerances must be specified that both protect the design objective as closely as possible and are practical and achievable with the materials and equipment available.

4.2.6.1 Tolerances—General Discussion

Tolerances for trackwork fall into four categories:

- Manufacturing/Fabrication Tolerances: The rolling, casting, machining, and finishing tolerances of track materials need to be appropriate to the intended service condition. On a light rail transit project, in virtually all cases, track materials should be of the highest standard/quality for new materials, and tolerances will hence be tighter than those used for ordinary freight railroad track. Rarely are there any tracks on a light rail system where second-quality or used materials are appropriate. Manufacturing and fabrication errors in finished products are difficult (and sometimes impossible) to correct in the field and can place a burden on the installation contractor attempting to construct acceptable track with inferior materials. See Chapters 5, 6 and 13 for additional discussion on this matter.
- Field Construction Tolerances: Track construction tolerances are most often specified with the use of new materials in mind. If used materials, such as relay grade rail, are employed, then construction tolerances may have to be less restrictive.
- Field Maintenance Tolerances: These represent the acceptable limits of wear and track settlement or misalignment for track systems components. After components are worn to this level, performance is considered to be sufficiently degraded such that wear and deterioration are likely to occur at an accelerated rate. At that time, maintenance should be performed to restore the system to a condition as close as possible to its new, as-constructed state.
- Field Safety Tolerances: These represent the levels beyond which the system is unsafe for operation at a given speed. The FRA Track Safety Standards are a well-known example. If track systems are permitted to degrade to an unsafe condition, performance

will be unsatisfactory, wear will be excessive, and the cost of restoration to a satisfactory state will be high. Either immediate corrective repairs, reduced speeds, or both are required once track has deteriorated to this condition.

In all cases, the degree of uncertainty associated with the measurement methodology should be considered. It must also be recognized that the geometric parameters of the track under load can be appreciably different than when it is not loaded.

4.2.6.2 Tolerances and Track Gauge

The reduced differential distance between track gauge and wheel gauge in transit systems governs the gauge tolerances for both. A suggested practice is to have a plus tolerance for track gauge and a minus (no plus) tolerance for wheel gauge, especially when the track gauge/wheel gauge freeplay is small by design. While both track gauge and wheel gauge typically have plus and minus tolerances, so as to avoid interference, the minus tolerance on track gauge and the plus tolerance on wheel gauge should be as close to zero as possible.

If performance of the system is to be as expected, it is equally important that the vehicle side of the wheel/rail interface be built to very specific dimensions and within tight tolerances. Achieving tolerances on wheelsets in new light rail vehicles is rarely an issue. Where projects have come to grief at that interface, the fault usually lies in a lack of coordination between the vehicle engineer and the track engineer on issues of wheel profile and wheel gauge.

4.2.6.3 Suggested Track Construction Tolerances

Transit track construction tolerances are more restrictive than conventional railroad standards. **Table 4.2.1** lists suggested track construction tolerances for the three general types of LRT track construction.

The following should be considered in developing tolerances to a particular project:

- Achieving accurate track gauge when constructing with concrete cross ties is much easier than when constructing with timber ties as the former are manufactured with the rail fastening assemblies included. Strictly speaking, the tolerances given for concrete tie yard track are unnecessarily tight; nevertheless, they are easily achievable.

- When considering minus tolerances for track gauge, consideration should be given to the freeplay between the track gauge and the wheels. The minus tolerance can be more liberal if AAR wheel gauge is used than if a transit wheel gauge is used.

- The tolerances given are generally independent of train speed. Embedded and direct fixation tracks in slow speed secondary tracks (such as in a shop) can reasonably use looser tolerances.

See Chapter 13, Article 13.2.3.4, for additional discussion concerning track construction tolerances.

Table 4.2.1 Track construction tolerances

Type of Track	Track Gauge (5)	Guard Rail Gauge (5)	Cross Level (5)	Construction Tolerances		Location Tolerances	
				Horizontal Alignment Deviation (1) (5)	Vertical Alignment Deviation (1) (5)	Horizontal Alignment Variable (6)	Vertical Alignment Variable (6)
Ballasted concrete cross ties (Main Line)	+/- 1/16" [+/-1 mm]	+1/8",-1/16" [+3,-1 mm]	+/- 1/8" [+/- 3 mm]	1/4" [6 mm] (2)	1/4" [6 mm] (3)	1/2" [13 mm]	1/2" [13 mm]
Ballasted timber cross ties (Main Line)	+/- 1/8" [+/-3 mm]						
Ballasted concrete cross ties (Yard)	+/- 1/16" [+/-1 mm]	+1/8",-1/16" [+3,-1 mm]	+/- 3/16" [+/- 5 mm]	3/8" [9 mm]	3/8" [9 mm]	1/2" [13 mm]	1/2" [13 mm]
Ballasted timber cross ties (Yard)	+3/16", -1/16" [+5, -1 mm]						
Direct Fixation	+1/8",-1/16" [+3, -1 mm]	+1/8",-1/16" [+3,-1 mm]	+/- 1/8" [+/- 3 mm]	1/4" [6 mm] (2)	1/4" [6 mm] (3)	1/4" [6 mm]	1/4" [6 mm]
Embedded	+1/8",-1/16" [+3, -1 mm]	+1/8",-1/16" [+3,-1 mm]	+/- 1/8" [+/- 3 mm]	1/4" [6 mm] (2)	1/4" [6 mm] (3) (4)	1/4" [6 mm]	1/4" [6 mm]

(1) Deviation is the allowable construction discrepancy between the standard theoretical designed track and the actual constructed track.

(2) Deviation (horizontal) in station platform areas shall be: zero inches [millimeters] toward platform, 0.125 inches [3 millimeters] away from platform.

(3) Deviation (vertical) in station platform areas shall be: plus 0, minus 0.25 inches [6 millimeters] or in conformity with current ADAAG requirements.

(4) Deviation at top of rail to adjacent embedment surface shall be plus 0.25 inches [6 millimeters] minus 0.

(5) Rate of change variations in gauge, horizontal alignment, vertical alignment, cross level, and track surface shall be limited to 0.125 inches [3 millimeters] per 15 feet [4.6 meters] of track.

(6) Variable is the allowable construction discrepancy between the theoretical mathematized and the actual as-built locations of the track. Tracks adjacent to fixed structures shall consider the as-built tolerances of the structures.

The data in Table 4.2.1 should not be confused with tolerances pertaining to track maintenance and track safety limits. Track maintenance limits that define allowable wear and surface conditions are not included in Table 4.2.1, as they should be developed with due consideration to the needs of a particular transit operating agency.

4.3 GUARDED CURVES AND RESTRAINING RAILS

It is customary in North American light rail track design to provide a continuous guard rail or restraining rail through sharp radius curves. The term "restraining rail" will be used throughout

this Handbook so as to avoid any confusion with either the guard rails positioned opposite a frog or the "emergency guard rails" that are often positioned between the running rails on bridges.

In addition to the discussion that follows here, readers are encouraged to consult two other documents on the topic of restraining rail that were produced by TCRP Project D-07—*TCRP Research Results Digest 82: Use of Guard/Girder/Restraining Rails* [7] and *TCRP Report 71: Track-Related Research—Volume 7: Guidelines for Guard/Restraining Rail Installation*.[8] An additional source of useful information is an article, "Testing Girder Rail on the MBTA" [9], a 2007 discussion in a web publication titled, *Interface—The Journal of Rail/Wheel Interaction*.

4.3.1 Functional Description

In a typical LRT installation, the restraining rail is installed inside the gauge line of the curve's low rail to provide a uniform flangeway. Restraining rail provides additional wheel steering action using the back face of the flange of the wheel that is riding on the inside rail of the curve. The inside wheel contact with the restraining rail takes some of the centrifugal force resulting from lateral acceleration and thereby reduces the lateral-over-vertical (L/V) forces of the outer wheel at the gauge corner of the outer rail.

Depending on the curve radius and the truck factors, the flangeway is typically 1 ¼ to 2 inches [32 to 51 millimeters] wide. The working face of the restraining rail bears against the back side of the flange of the inside wheel, guiding it away from the centerline of track and reducing the lateral contact force between the outside wheel's flange and the outer rail of the curve. This essentially divides the lateral force between two contact surfaces. Experience shows that this greatly reduces the rate of lateral wear on the high rail. (Curiously, the computer modeling that is the basis of *TCRP Report 71*, *Volume 7*, concluded that curves without restraining rail should have less wear than curves with restraining rail. As of 2011, this difference between theory and actual practice had not been reconciled.) Restraining rail also, by increasing the rolling resistance force applied along the inner rail, counteracts the tendency of the inner wheels to move ahead of their mates on the opposite end of the axle. This encourages backwards slippage of the inner wheels, rotates the truck in the direction of the curve and thereby reduces the angle of attack between the wheel flange and the outside rail. In all cases, the use of restraining rail in a curve will reduce the tendency of the leading outside wheel to climb the outer rail, thereby preventing possible derailments.

4.3.2 Theory

TCRP Report 71, *Volume 7*, describes two restraining rail philosophies that TCRP Project D-7 investigated:

- Philosophy I—"Shared Contact." This configures the system (track gauge, wheel gauge, wheel profile, and flangeway width) so that simultaneous "shared" contact occurs between both the outer rail and the front of the flange riding that rail and the restraining rail and the back of the wheel riding on the inner rail.

- Philosophy II—"No High Rail Contact." This configures the system so that no contact occurs between the flange of the outer wheel and the gauge face of the outer rail. Effectively, all lateral loading and steering action occurs at the working face of the

restraining rail. (Note that a small amount of loading would still be carried by surface friction between the tops of the rails and the treads of the wheels, but that is so small that it is usually neglected.)

A third philosophy, not directly addressed by *TCRP Report 71*, *Volume 7*, could be described as the following:

- Philosophy III—"No Routine Restraining Rail Contact." This configures the system so that routine contact occurs only between the flange of the outer wheel and the gauge face of the outer rail. The restraining rail is engaged only in the event that either (1) the outer wheel has begun to climb the outer rail or (2) the combination of wear on both the outer rail's gauge face and the flange of the wheel allows contact to occur at the working face of the restraining rail due to the outward shift of the axle. Note that in the case of an incipient derailment, a Philosophy III restraining rail might be properly called a "guard rail" since, like the guard rail opposite a frog, it is engaged only when actually needed to prevent a mishap.

Note that Philosophy I represents an idealized condition. It presumes that the system can be perfectly configured and ignores the realities of tolerances for fabrication, construction, and maintenance on both the track side and the vehicle side of the wheel/rail interface. Because of those factors, it is rarely possible to achieve Philosophy I during initial construction. Instead, what is usually done is to build the system so it more or less matches Philosophy II and allow it to "wear in" to a Philosophy I condition. This typically comes about through wear on the working face of the restraining rail. If track gauge is less than perfectly uniform, achieving equilibrium may require some wear on the gauge face of the outer running rail. Even once a shared contact condition is achieved, not all wheel sets will contact both the restraining rail and the outer running rail. Wheels with worn flanges will sometimes contact only the restraining rail while new wheel sets may contact only the outer running rail.

Philosophy III is most commonly seen on European light rail operations, particularly those in Germany. The reason for this is contained in the German federal regulations concerning tramways and light rail transit, commonly known as "BOStrab." BOStrab is a contracted form of *Verordnung über den Bau und Betrieb der Straßenbahnen,* which means *Regulations on the Construction and Operation of Street Railways.* BOStrab specifically prohibits the restraining rail configurations labeled above as Philosophy I and Philosophy II. The reasons for this restriction are unclear, but what is clear is that BOStrab is distinctly at odds with conventional North American practice in this matter. However, as is discussed in Chapter 2, Article 2.5.5.4, European rail vehicle designers and manufacturers, who are used to working under BOStrab, may object to the use of restraining rail, particularly if configured as per Philosophies I or II.

Each of the options above has its adherents, but, for trackwork practitioners who prefer restraining rail, Philosophy I is the most popular. *TCRP Report 71*, *Volume 7*, concluded that Philosophy I does (on average at least) dramatically reduce lateral loading of the rails and hence can be instrumental in preventing flange climb derailments. Perhaps notable is the fact that *TCRP Report 71*, *Volume 7*, asserts that the lateral force exerted on the restraining rail under Philosophy II is greater than the lateral force that would be exerted on the outer running rail without any restraining rail. The reason for this difference is not clear. *TCRP Report 71* also notes that lateral force on the outer rail is dramatically reduced (less than half) with shared

contact and that contact on only the restraining rail results in a higher lateral force than curves without restraining rail. This difference is unexplained but may be due to slight differences in the angles of attack between the restraining rail and the outer running rail. *TCRP Report 71*, *Volume 7*, states that the angle of attack is best with no restraining rail, but this assertion is counterintuitive and contrary to the results indicated by Nytram plots. Since *TCRP Report 71*'s simulation parameters for track gauge, wheel gauge, and flangeway width are unclear, this assertion requires more investigation.

The work that led to *TCRP Report 71*, *Volume 7*, relied heavily on field testing that was performed at the Transportation Test Center in Pueblo, Colorado, in the early 1980s. Those tests were performed on the test center's "Tight Turn Loop," which has a 150-foot [45.7-meter] radius for a full 360 degrees of arc. Tests were conducted both with and without restraining rail using a then-experimental heavy rail transit vehicle known as the "State of the Art Car" (SOAC). Track gauge and the restraining rail flangeway were configured so as to match Philosophy II described above. Unfortunately, surviving documentation does not reveal several key parameters including the width of the restraining rail flangeways, the track gauge, the type of wheels on the SOAC, and other factors. What is clear is that the SOAC bears little resemblance to contemporary light rail vehicles, particularly low-floor and partial low-floor cars. The authors of this Handbook believe that additional instrumented field testing using contemporary light rail vehicles is appropriate and may well be essential to reconciling the differences between North American and European perspectives concerning restraining rail.

4.3.3 Application Criteria

Restraining rails have been commonly applied on virtually all legacy rail transit systems (both light rail and heavy rail) in North America for well over a century. However, the thresholds at which restraining rails are applied varies greatly from system to system, Some transit agencies guard any curves with radii less than 1200 feet [365 meters], while others do not guard curves with radii larger than 300 feet [91 meters]. *TCRP Report 71*, *Volume 7*, recommends radius thresholds for restraining rail that vary depending on the type of vehicle and the track classification. Rather than condensing that information here, users of this Handbook are encouraged to scrutinize *TCRP Report 71* in its entirety and make decisions based on the specific characteristics of their project.

4.3.3.1 Non-Quantifiable Considerations for Restraining Rail

While designers are fond of exact criteria based on formulae, not all design can be that precise, and restraining rail is a key example of that. Non-quantifiable factors to consider with respect to the application of restraining rail are the following:

- Lower train speeds reduce both lateral acceleration and the consequent lateral forces between the wheel flanges and the rail. This should reduce the lateral component of the L/V ratio, decreasing the probability of a wheel flange climb derailment. However, computer simulations conducted as a part of TCRP Project D-7 (results published in *TCRP Report 71*, *Volume 7*)[8] don't fully support this premise. Nevertheless, field observations strongly suggest that unguarded curves of very tight radius can be safely operated at slow speeds while operation on the same curves at higher speeds presents a high risk for a flange climb derailment. Additional research, including field testing, is likely warranted.

- Frequently used tracks tend to develop a polish on the wheel/rail contact surfaces, which obviously reduces friction. Informal field observations suggest that tracks with these shiny rails that are both sharply curved and frequently used can be successfully operated without restraining rail and also with relatively little noise. By contrast, operation over rusty rail at the same curve radius can be very noisy and has a high probability of the wheel flange climbing the outer rail. *TCRP Report 71*, *Volume 7*, stipulates that no restraining rails are required if the coefficient of friction (μ) between the wheels and the rail can be kept under 0.4. (As a point of reference, the usual reference manual value for μ between clean, smooth, and non-lubricated steel surfaces is 0.8.) Rusty wheels or rails, wheels that have been freshly trued, or rails that have been freshly ground will result in higher values of μ. Lubrication of both the gauge face of the outer rail and the top of the inner rail of infrequently used (and hence rusty) sharp curves has been shown to decrease friction sufficiently to permit safe operation.

- It is notable that the aforementioned experiment with the SOAC was performed well over a decade prior to the introduction of contemporary top-of-rail friction modifiers and about two decades before the introduction of modern on-board lubrication systems such as those described in Chapter 2, Article 2.8. Repeating the SOAC experiments with both modern LRVs and modern rail lubrication methods could produce substantially different results.

- The research behind *TCRP Report 71*, *Volume 7*, concluded that restraining rail can prevent flange climb derailment that might otherwise occur because of track perturbations such as low joints and horizontal misalignments. In essence, poorly maintained track can derive more benefit from restraining rail than well-maintained track. It can therefore be inferred that rigid trackforms, such as embedded and direct fixation track, which are less likely to suffer misalignments, have less need for restraining rail than ballasted track. Moreover, the authors of this Handbook suspect that BOStrab's prohibition of restraining rail is based in part on an expectation that track perturbations will never be allowed to reach the levels suggested in *TCRP Report 71*, *Volume 7*. In that regard, it must be noted that maintenance activities at European transit agencies are typically better funded than at transit authorities in the United States.

- An LRT system using wheel flanges that are short, such as those that are common on legacy streetcar lines, will have a greater need for curve guarding than one that uses railroad-type wheels with tall flanges. This is because the lateral wheel loading is distributed over a narrower contact band along the side of the rail head thereby increasing contact stresses and resultant wear on both wheel flange and rail. Transit systems that use short flanges usually have a characteristic stepped wear pattern on the high rail of their curves.

- Per *TCRP Report 71*, *Volume 7*, a rail vehicle wheel with an angle on the front face of the flange of less than 75 degrees will have a greater need for restraining rail than one with that optimal angle.

- As of 2010, there did not appear to have been any studies of the optimal angle on the back face of the wheel where it interfaces with the restraining rail. Notably, the ATEA

standards recommended guard face angles that varied by the curve radius and most likely mimicked the angles to which restraining rails naturally wore in service. It should be noted that the contact point between an inside restraining rail and the back of the wheel usually occurs appreciably ahead of a vertical projection from the centerline of the axle. That location varies with both the curve radius and the angle of attack.

- In theory, a system with vehicles that are equipped with a self-steering radial truck design should not need guarded track.

- Conversely, LRT systems using LRV trucks with independently rotating wheels, which have no inherent steering capability, could possibly derive significant benefit from restraining rails since they can, if configured appropriately, correct extreme truck skewing through curves.

Much of the conventional wisdom concerning restraining rails was developed nearly a century ago, when virtually all rail vehicles had solid, non-resilient wheels and solid axles. Resilient wheels and independently rotating wheels existed, but were experimental oddities. Now that both resilient wheels and independently rotating wheels are common, the authors believe that additional research is needed to optimize the wheel/rail interfaces where restraining rail is used, especially when used by vehicles using trucks equipped with modern suspension systems and wheels. Since there are dozens of modern truck designs and nearly as many designs of resilient wheels, a single set of criteria concerning restraining rail may not be possible. Track designers whose project includes a mixed vehicle fleet may need to consider the needs of each vehicle.

4.3.3.2 Longitudinal Limits for Restraining Rail Installations

Curve guarding does not usually terminate at the point of tangency of a curve. Instead it extends some distance into the adjacent tangent track. This distance depends on a number of factors, including the resistance to yaw of the vehicle's suspension system. The conservative designer will extend the restraining rail a distance no less than one axle spacing of a truck into the tangent track, typically rounded up to about 10 feet [3 meters]. When the curve is spiraled, the beneficial effects of guarding typically end long before the spiral-to-tangent location. In such cases, curve guarding can usually be terminated at the end of the spiral. Exceptions can be considered in cases where an unusually long spiral is used or when a compound curve condition exists with one curve segment guarded and the other not.

The criteria for beginning curve guarding on the entry end of the curve are typically the same as for the exit end, accounting for the possibility of occasional reverse running train operation. As a guideline, the minimum guarding should begin at the tangent-to-spiral location of a spiraled curve so that the vehicle trucks are generally square to the track well before entering the portion of the spiral equal to the threshold radius for guarding.

On some transit systems, the design criteria for projection of a restraining rail into adjoining tangent track are as much as three times the figures cited above. In at least one case, this was a direct reaction to problems encountered with derailments of a very early model of LRV in the 1970s. The actual cause of those derailments may have been that the truck's equalization capability was not a match for the severity of the track twist, but extended restraining rail was the

nominal solution. More recent evidence suggests that no benefit is obtained by extending restraining rail more than about 10 feet [3 meters] into tangent track.

4.3.4 Curve Double Guarding

Some transit agencies, notably the legacy systems, "double guard" extremely sharp curves, placing a restraining rail adjacent to the high rail as well as the low rail. These double-guarded installations are designed to counter the tendency of the second axle on a truck to drift toward the low rail.

That motion occurs because the wheels on the inner rail, having a shorter distance to travel than those on the outer rail, are constantly moving ahead of their mates on the opposite end of each axle. As a result, the truck rotates in a direction opposite to the orientation of the curve, and the flange on the inside wheel of the trailing axle usually bears against the low rail. This brings the angle of attack on the outside wheel of the leading axle to a maximum. Depending on the stiffness of the journals of the trucks, the truck can actually assume the plan shape of a parallelogram. This condition is, of course, unstable, and something must slip to restore equilibrium. In a worst case, the leading outer wheel climbs the rail and derailment occurs, but the usual case is that the wheels on the inner rail skip backwards, thereby briefly rotating the truck in the direction of the curve. This backwards skipping is plainly visible on extremely tight curves, such as those that are common on legacy streetcar lines. This roll-skip-roll-skip process is repeated through the full length of the curve. Concurrent with the skip backwards is a lateral slip and rotation across the rail head at all four wheels as the truck rotates. This combined transverse and longitudinal slippage on the rail head—typically called "stick-slip"—is actually the source of most curving noise, especially on the low rail, where the slip distance is longer.

In a double-guarded curve, the restraining rail placed alongside the outside rail prevents the truck from fully rotating to the point where the inner wheel on the trailing axle is in hard contact with the inner rail. Instead, the back of the trailing axle's outer wheel is bearing on the outer restraining rail. This reduced truck rotation thereby reduces the angle of attack at the leading outside wheel. It also reduces the magnitude of each cycle of stick-slip, since the inner wheels don't need to skip backwards as far to restore equilibrium. The outer restraining rail, by essentially pulling the trailing axle away from the inner rail, also keeps the truck reasonably square to the track, with both axles closer to a radial orientation, and assists in keeping the truck frame rectilinear. It also reduces the amount of forward motion that occurs before wheel/rail slippage occurs, effectively reducing the amplitude of each cycle of stick-slip.

In superelevated, sharp radius curves where the vehicle speed is reduced, the vehicle truck may tend to hug and climb the low rail. The outer restraining rail reduces this wheel climb potential.

As a guideline, a typical threshold for consideration of double-guarded track is for curves with radii of 100 to 125 feet [30 to 38 meters].

4.3.5 Restraining Rail Design

In North America, curve guarding on traditional street railway systems was most frequently achieved using a girder guard rail section somewhat similar to the 56R1 section illustrated in

Figure 5.2.5 of this Handbook, particularly for track embedded in pavement. For open track design, such as ballasted or direct fixation track, a separate restraining rail mounted alongside the running rail is more commonly used. The restraining rail can be machined from a section of standard tee rail, which can be mounted either vertically or horizontally. Specially rolled or fabricated steel shapes are also used, as described and illustrated in Chapter 5, Article 5.3.

4.3.5.1 Restraining Rail Working Face Angle

Similar to the gauge face of the outer rail of a curve, the "working face" of a restraining rail on the inside of a curve tends to assume an angle to the vertical as it wears in. This is because the leading edge of the wheel, where it contacts the restraining rail, is non-tangential to the rail. The wheel also is contacting the restraining rail with two radial surfaces—the cross-sectional shape of the flange and the diameter of the wheel. This wear tends to stabilize at an angle of 10 to 15 degrees from the vertical, depending on the radius of the curve. The potential problem is that by the time the working face has reached the optimum angle, wear may have widened the flangeway some appreciable amount larger than its optimal dimension. For this reason, some restraining rail designs machine the working face of the guard at the time of fabrication, so no metal needs to be worn away before the optimal angle is reached. The former ATEA girder guard rails were manufactured with a 20-degree angle on the working face of the guard since that was optimal for the extremely tight minimum radii used on many legacy streetcar lines. The AREA girder guard rails last rolled in the United States in the 1980s had a 16-degree working face angle. By contrast, most European groove rails have an angle equivalent to roughly $9^{o}30'$. Notably, those ATEA and AREA guard face angles, which effectively are service-proven designs, may be more severe than a Nadal analysis would permit for gauge face wear on the outer rail of the curve. This dichotomy is a subject worthy of more detailed investigation.

It generally is not necessary to consider a vertical angle on a restraining rail along the outer rail of the curve.

4.3.5.2 Restraining Rail Height

Restraining rail designs typically also project above the plane of the running rails. The guards on American girder guard rails were ¼ to $^{3}/_{8}$ inch [6 to 10 mm] above the top of rail. Some designs of separate restraining rails are as much as an inch above the running rails. The reason for this is to intersect more of the vertical back face of the wheel and not just the angled back of the flange. Restraining rails that project above the running rails also reduce any tendency of the wheel to climb the restraining rail. Notably, the lip on most European groove rails is typically 5 to 10 mm [0.2 to 0.4 inch] below the top of rail, which significantly reduces their possible effectiveness as a restraining rail.

Restraining rails that project a substantial distance above the top of the running rails may interfere with some equipment on the light rail vehicle trucks, particularly magnetic track brakes. Elevated restraining rail positions can also interfere with hy-rail, rubber-tired, maintenance-of-way vehicles by lifting the rubber tires that propel the vehicle along the rail. This action may lift the hy-rail gear on the rear of the vehicle and result in a derailment. Some municipalities may object to elevated restraining rails in embedded track on the grounds that they might interfere with snow plowing; however, that is unlikely to actually cause problems unless the guard is significantly

more than the usual ¼ inch [6 mm] above the running rail, particularly if one looks objectively at the typical construction and maintenance tolerances for the pavement on urban streets.

Both the height and the working face angle of the restraining rail should be considered when determining the most appropriate flangeway width.

4.3.5.3 ADAAG Considerations for Restraining Rail

When restraining rails are used in a pedestrian path, care must be taken to comply with ADAAG requirements. This can restrict the height of the restraining rail above the running rail. Article 303 of ADAAG stipulates that the maximum permissible vertical bump in an accessible path is ¼ inch [6.4 mm]. An additional ¼ inch is acceptable provided it is ramped. While this requirement was most likely written with building doorsills in mind, it technically applies to any location along an accessible path, including crossing a railway track. Since there is no way to ramp across an open flangeway, this effectively limits the height of any restraining rail located along a pedestrian path to ¼ inch above the top of the running rail. However, vertical wear on the running rail must also be considered, since such wear would have the effect of increasing the height of the restraining rail. Viewed collectively, these considerations suggest that restraining rail in a pedestrian route should be no higher than level with the top of the new running rail. That way, once rail head wear occurs, the installation will still be in compliance with ADAAG.

Note that the limitation stated above applies only to a designated pedestrian route, such as a crosswalk. Outside of such designated routes, the configuration of the restraining rail might be different, subject to any other constraints. For example, for track that is embedded in a mixed traffic lane of a public street, the restraining rail height should not present a hazard to vehicular traffic, especially bicycles and motorcycles. As noted above, the designer should consider the maximum wear condition when assessing the height difference.

ADAAG restricts the width of rail transit system flangeways in an accessible path to no more than 2 ½ inches [63.5 mm]. Since that dimension is greater than any expected restraining rail flangeway, that requirement is not an issue. However, one topic that ADAAG does not address is flangeway depth. Small wheels on mobility aids can, when crossing a flangeway on a skew, easily spin and then drop down into the flangeway, possibly trapping the wheel. For this reason, it is strongly recommended that the restraining rail flangeways crossing an accessible path be no deeper than about 2 inches [50 mm] below the top of the running rail. If the restraining rail is machined from a vertically mounted tee rail, the open flangeway can be filled with an elastomeric grout up to the desired flangeway depth. This strategy also has the advantage of sealing the open flangeway, thereby excluding moisture that could penetrate and damage the track structure.

4.3.6 Omitting Restraining Rails—Pros and Cons

The use of restraining rails is far from universal. Several of the light rail systems built in North America since 1980 use no restraining rail at all, even on the sharpest curves. In part, that can be explained by the fact that those systems were designed by persons with railroad trackwork backgrounds where restraining rails are virtually unknown. Nevertheless, those systems appear to function satisfactorily albeit with increased rail wear and slower operating speeds. It's notable that if the flangeways on embedded tracks on those systems are not wide enough, the roadway

pavement can be abraded and damaged by the backs of the wheels as they briefly act as a de facto restraining rail.

Arguably, in open trackforms (i.e., ballasted and direct fixation track) it may be both easier and more cost-effective to replace the high rail more often than it is to go to the extra expense and trouble of installing a restraining rail adjacent to the low rail. It has also been argued that the presence of a restraining rail can compromise the signal system's ability to detect a broken running rail by providing an alternative path for the signal current. The details of the fastening system for a restraining rail also can be additional locations for stray current leakage.

Due to BOStrab regulations, the use of restraining rail is uncommon on European LRT and tramway lines. Despite the near universal use of groove rail in European embedded track, open trackforms using tee rail on the same tramway lines will very frequently have no restraining rail on even the sharpest curves. Moreover, European tramways typically set track gauge very precisely so as to avoid any routine contact between the backside of the wheels and the lip on the groove rail, depending entirely on contact on the front of the wheel flanges for all steering action. Perhaps for this reason, most European groove rail sections have lips that are relatively thin. For example, the popular 59R2 section has a tram that is only 15-mm [0.59-inch] thick. The tram on the similar 60R2 section is only 21-mm [0.83-inch] thick.

Those dimensions contrast sharply with the tram on the former ATEA's girder guard rails, which was 1-$^{15}/_{32}$-inch [37-mm] thick. After girder rail was no longer rolled in the United States, several North American light rail systems began using the European groove rails. However, they did not yet appreciate the differences between the American and European designs and presumed the latter would perform in the same manner as the former. There was appreciable concern when it was first noticed that the lip on the European groove rails wore dangerously thin in a very short period.

As will be discussed in Chapter 5, there are two European groove rail sections that provide a guard of appreciable heft; however, these guards are used by relatively few tramway systems compared to the more popular sections. Instead, following the letter of BOStrab, the general philosophy in most of Europe appears to be that the lip, or guard, on all groove rail sections is something that should only come into play when either the outer rail's gauge face wear has reached a condemning limit or derailment is imminent. However, it should also be noted that, in general, standards for maintenance of light rail tracks are much higher in Europe than they are in North America. Moreover, European transport agencies are routinely provided with the budget necessary to both construct and maintain tracks to high standards, including replacement of worn rail. Few transit agencies in North America are so well funded. For this reason alone, direct comparisons between European and North American transit trackwork design principles—including application criteria for restraining rail—can be very misleading.

As noted in Chapter 2, the general disuse of restraining rail in Europe has led European-based carbuilders to reduce the mass and stiffness of the vehicle axles in pursuit of reduction of unsprung vehicle mass. This reduces the vehicle's capacity to accept the forces imposed by restraining rail contact. Vehicle engineers who are schooled in European practice may therefore strongly oppose the use of Philosophy I and II restraining rails. The track engineer wishing to use restraining rail on a new project may need to build a strong business case to justify the installation

on a life cycle cost basis, including vehicle-related procurement and operation and maintenance costs.

4.4 TRACK SUPPORT MODULUS

Railway track acts as a structural element that undergoes stress and strain as a vehicle passes over it. The rails, rail fasteners or fastenings, cross ties, ballast, subballast, and subgrade are each a component of the track structure. Each undergoes some deflection as the wheel passes.

The question of how the track structure reacts to wheel loads was studied as early as 1914, when a committee of what was then called the American Railway Engineering Association, chaired by Professor Arthur Newell Talbot of the University of Illinois, commenced investigations that led to the first definitive work on this subject. This Handbook provides sufficient information to design track; for additional reference, the designer is advised to study either the *Talbot Reports* of 1920 (available from AREMA in reprinted form) or Dr. William W. Hay's textbook, *Railroad Engineering*, both of which provide more detailed explanations.[2],[5] Additional resources include AREMA's *Practical Guide to Railway Engineering*. However, the reader is cautioned that engineering standards developed for freight railroad applications are frequently incompatible with the requirements of rail transit design, and direct application of information from these references should only be undertaken with due consideration of the differences in vehicles, loadings, and the trackway environment.

Track modulus is an important subject, using complex mathematical calculations to analyze ballasted track as a structure. This analysis can determine appropriate rail weights, cross tie size, cross tie spacing, and ballast depth, as well as the need for subballast and any special subgrade preparation. Similar mathematical calculations can be undertaken for direct fixation and embedded trackforms.

The track modulus factor value (typically represented by the symbol □) established in this article is a requirement of track design and one of the variables used in the calculations for ballasted track structural design (see Article 4.5.3) and direct fixation track structure design (see Article 4.6.3). In addition, track modulus is a parameter found in many of the calculations used by noise and vibration engineers when considering wheel impacts, contact separation, and vibration.

4.4.1 Modulus of Elasticity

Ballasted track is often characterized as a beam supported on a continuous series of springs. Track modulus can be defined simply as the amount of deflection in these springs for a given wheel load. The greater the deflection, the lower the modulus. Conversely, a track with little deflection has a high modulus, which is generally considered important for ride quality and good serviceability in ballasted track. Most of the deflection in ballasted track results from deformation of the ballast and subgrade, with only minor deflections resulting from rail and cross tie compression. In order to minimize deflections, the track should have a deep section of well-compacted ballast and subballast with a sound, compacted, well-drained subgrade. This is crucial if total rail deflections for ballasted track are to be kept under the ¼-inch [6-millimeter] limit suggested by AREMA.

In direct fixation track, the track modulus is typically much higher, because the rail fasteners are made of elastomer with relatively high stiffness. In direct fixation track, the track designer is more frequently challenged to engineer a lower modulus into the track where possible, while still retaining required levels of gauge restraint and corrugation control. Reducing track modulus is desirable to the degree that it mitigates impact loading of the track and generation of high-frequency vibration. Soft direct fixation fasteners with elastomer in shear are available for providing a rail support modulus approximating that of ballasted track. However, as of 2010, such fasteners were somewhat more expensive than direct fixation fasteners of normal stiffness.

The modulus of embedded trackforms is typically much higher than the modulus of open trackforms since there are very limited voids into which deflection can occur. When rails with elastomer/rubber boot encapsulation are embedded directly into concrete pavement or the bare rail is placed into other types of elastomeric embedment material, small but detectable amounts of deflection will result. However, it must be understood that elastomers cannot compress or deflect unless there is some void into which they can deflect. Solid elastomers are usually considered to be incompressible, and some amount of unloaded free surface area is required to allow the elastomer to deflect under load. The ratio of one of the loaded surfaces to the free surface is referred to as the "shape factor." High shape factor produces high stiffness.

The explanation below deals with ballasted track modulus, which can be determined using the this equation from Professor Talbot's work:[5]

$$P = -\mu y$$

where

P	is the upward force on the rail per unit length
μ	is a factor determining the track stiffness or "modulus of track" given in units of pressure
y	is the vertical deflection measured at the base of rail

The modulus of the track is defined as the vehicle load per unit length of track required to deflect the rail one unit. An example follows:

Assume that on a track with cross tie spacing of 30 inches [762 mm], a wheel load of 20,000 pounds [88,964 newtons] causes a track vertical deflection of 0.375 inches [9.5 millimeters]. The force P required to deflect the track 1 inch [or 1 millimeter] is

$$\frac{P}{20{,}000} = \frac{1}{0.375} \qquad \left[\frac{P}{88{,}964} = \frac{1}{9.5}\right]$$

$$P = 53{,}333\ \text{lbs./in.} \qquad [P = 9{,}365\ \text{N/mm}]$$

The track modulus is equal to the force per unit of track length required to deflect the track by one unit, i.e., 1 inch [or 1 millimeter]. In this example, with cross tie spacing at 30 inches [762 millimeters], the track modulus is

$$53{,}333/30 = 1{,}778\ \text{lb/in/in} = \mu$$

$$[9{,}365/762 = 12.3\ \text{N/mm/mm} = \mu]$$

The above analysis assumes that either the desired rail deflection is known or that maximum rail deflection is the primary criterion for the track design. Increasing the track modulus will dramatically reduce the bending moments in the rail. However, the higher modulus will also increase pressures on the ballast and subballast by directing more of the wheel load to the track support directly under the load. The ballast and subballast must be designed with the capacity to support those loads, as noted in the next section.

Note that the variables used in calculating track modulus consider the support properties of a single rail and the loading of a single wheel, to simplify calculations. The load and deflection of a single rail applies equally to the track structure, since both the load and the stiffness are doubled. The effects of differential rail loading due to unbalance on curves are not specifically considered in the analysis but should be accounted for, along with impact, in determining worst-case service loads.

4.4.2 Track Stiffness and Modulus of Various Track Types

The stiffness of rail, fastenings, and supporting structure determines the "Stiffness of Track," whereas "track modulus" is concerned only with the support condition of the rail. (See *TCRP Research Results Digest 79*[6] for additional discussion on this point.) The types of track encountered on an LRT system—ballasted, direct fixation, and embedded—have a wide range of stiffness and track modulus because the components of each track substructure are dramatically different. Ballast provides the most flexible track structure support, while embedded track is usually the stiffest, with the highest track modulus value. Resilient direct fixation track can provide a wide range of stiffness by selection of rail fastener with engineered values of stiffness.

4.4.2.1 Ballasted Track

The track modulus can be derived on a segment of existing ballasted track by measuring its deflection under load and calculating the modulus in accordance with the Talbot principles shown in Article 4.4.1. However, note that the Talbot formula is based upon track deflection due to a single axle load. If deflection is measured under a two-axle truck, an adjustment must be made because the nearby second wheel also contributes to local track deflection. Professor Arnold D. Kerr provides a method to adjust the modulus calculation to account for the weight of an adjoining second axle.[10]

In many cases, the maximum rail deflection is not known or the maximum rail deflection is to be estimated for a given track structure that is yet to be built. The latter condition is frequently encountered in ballasted track design.

The track modulus can be estimated considering the cross tie type and size, structure depth of subballast and ballast, type of ballast rock or stone, and the cross tie spacing. As a guideline, the track modulus with the track structure described can be expected to be in the following ranges:

- 1500–2500 psi [10–17 N/mm^2]: track - 18 inches [457 mm] depth of subballast and limestone ballast, timber ties spaced at 22 inches [558 mm].
- 2500–3500 psi [17–24 N/mm^2]: track - 22 inches [558 mm] depth of well-compacted subballast and heavy stone ballast, timber ties spaced at 22 inches [558 mm].

- 3500–5000 psi [24–34 N/mm^2]: track - 24 inches [609.6 mm] depth of well-compacted subballast and heavy granite ballast, timber ties spaced at 20.5 inches [520 mm].
- 5000–9000 psi [34–62 N/mm^2]: track - 24 inches [609.6 mm] depth of well-compacted subballast and heavy granite ballast, concrete ties spaced at 28 inches [711 mm].

The type of fastening system between the cross ties and the rails can affect track stiffness, although apparently little research has occurred in that matter. Timber tie track using elastic rail clips will generally be stiffer than an otherwise identical track using traditional cut spikes.

Track modulus has been known to vary and lose vertical support with an increase in applied load; that is, modulus under a 70-ton [63,500-kilogram] railroad freight car may have a lesser value when measured under a 100-ton [90,700-kilogram] railroad car. If this occurs, it is likely the result of overstressing the subgrade to the point that it deflects non-linearly. This is unlikely to occur under rail transit loadings except in cases where the subgrade soils are especially weak and compressible.

A higher track modulus results in higher stress concentrations on the ties and ballast directly beneath the wheel than does a low-modulus track, which distributes more of the wheel load to adjoining ties. Concrete ties, which always increase the track modulus, therefore require a stronger ballast/subballast foundation than timber ties, and the track section must be designed accordingly. Provision of a stronger foundation generally entails a deeper ballast and subballast layer, installation of a geogrid, or other measures to distribute the load over a broader area of the subgrade.

4.4.2.2 Direct Fixation Track

Unlike ballasted track, the track component deflections and elastic properties of direct fixation track are generally known. In direct fixation track, the vertical deflection occurs in the

- Bending of the rail
- Elastomer portion of the direct fixation fastener
- Flexure of the direct fixation slab at the supporting subbase materials for at-grade installations.

The track modulus of direct fixation track is determined by establishing the nominal spring rate of the elastomeric component of the direct fixation fastener. The spring rate is controlled by the ability of the elastomer to bulge, both along the free area at the periphery of the fastener (where it is usually exposed) and into the recesses within the fastener body. Manufacturers can control the spring rate within fairly narrow bands by customizing the sizes of these recesses, which are typically visible on the underside of the fastener body.

Elastomer vertical static spring rates vary widely. Three popular spring rate ranges are

- 50,000 to 80,000 lb/in [8,800 to 14,000 N/mm]—this range is highly resilient.
- 90,000 to 140,000 lb/in [15,800 to 24,500 N/mm]—this range is standard.
- 240,000 to 320,000 lb/in [42,000 to 56,000 N/mm]—this range is stiff.

It is worthwhile to note that the spring rate of a direct fixation rail fastener is virtually never linear from a condition of zero load up to maximum service load. Instead, due to the elastic behavior of elastomers under loading, a plot of load versus deflection would be a curve. The nominal spring rate of the fastener would be the slope of a line that is tangent to the load deflection curve within the zone of the actual service loading.

Selection of the proper fastener stiffness should take into consideration wheel loads, fastener spacing, degree of route curvature and unbalance, and noise and vibration issues. Heavier axle loads require a higher track modulus, as track deflection must be limited to values that will not cause fatigue in the fastener elastomer. In addition, low spring rates in some types of fastener designs will permit the rail to rotate outward under lateral loads due to differential compression of the elastomer layer, causing dynamic gauge widening that may reach undesirable levels. While the AREMA *Manual for Railway Engineering* currently does not specify maximum vertical rail deflections for direct fixation track, normal practice is to limit the deflection to around ⅛ inch [3 mm] under normal service loads.[4] Some fastener designs require even lower deflections, so the designer is encouraged to contact technical representatives of fastener manufacturers for input on this criterion.

For additional information on direct fixation rail fasteners, refer to Chapters 5 and 7 and also to *TCRP Report 71*, *Volume 6: Direct-Fixation Track Design Specifications, Research, and Related Material* (TCRP Project D-7, Task 11).

Fastener spacing, like the spacing of ties in ballasted track, is a factor in the modulus of direct fixation track; a common spacing for fasteners is 30 inches [762 millimeters]. The fastener stiffness divided by the fastener spacing gives the rail support modulus:

$$\mu = \frac{k_f}{a}$$

where

- u is the rail support modulus, lb/in/in [kN/mm/mm]
- k_f is the fastener stiffness, lb/in [kN/mm] as confirmed by testing
- a is the fastener spacing, in [mm]

The following example uses a fastener stiffness of 100,000 lb/in (17.51 kN/mm) at a spacing of 30 inches (760 mm):

$$\mu = \frac{k_f}{a} = \frac{100{,}000\ lb/in}{30\ in} = 3{,}333\ lb/in^2$$

$$\left[\mu = \frac{k_f}{a} = \frac{17.51 kN/mm}{760\ mm} \times \frac{1000 N}{kN} = 23.04\ N/mm^2\right]$$

The dynamic spring rate of most natural, rubber-based, elastomeric, direct fixation rail fasteners is 10% to 100% higher than the static spring rate due to the material relaxation properties of the elastomer. Dynamic spring rate can be most easily visualized by considering that the elastomer

has not fully recovered when the next wheel load is applied. Low ratios of dynamic to static stiffness are achieved with natural rubber fasteners with low shape factor or in shear.

The net effect of the dynamic spring rate being higher than the static spring rate is that the rail support modulus under normal train speeds is higher than static fastener tests would indicate. It is important that both static and dynamic fastener spring rate testing be conducted on rail fasteners, because the dynamic stiffness is a more accurate indicator of the fastener's performance under traffic than the static stiffness. The Dynamic/Static Stiffness Ratio (D/S Ratio) is dependent on the type of elastomer material used as well as the configuration of voids in the elastomer. In general, a lower D/S Ratio is desirable because the resulting lower track support modulus reduces impact loads and vibration forces transmitted to the invert. However, elastomers with higher D/S Ratios may offer greater damping of the dynamic resonance of the rail on the fastener and the so-called pinned-pinned mode of rail bending, which has been implicated as one cause of rail corrugation.

4.4.2.3 Embedded Track

The track support modulus for embedded track is very dependent upon the design of the immediate rail support (such as elastomer embedment or elastomer rail boots) and the underlying base slab.

For ballasted track that has an overlay of some sort of pavement material (known as "paved track," as distinct from embedded track), the track modulus will be in the range of ballasted track, 1500–4500 psi [10.3 to 31.1 kN/mm^2]. See Article 4.4.2.1 for ballasted track modulus values. If the pavement extends down into the tie crib areas, and, especially if the pavement is constructed underneath the ties, the track structure behaves more like a slab. Ballasted track equations are not valid for the latter case.

Many of the embedded track designs constructed in the 1980s and 1990s were essentially direct fixation trackwork installed in open troughs formed in an underlying concrete slab. For such designs, where the trough infill material provides little or no structural support, or where only elastomeric side pieces are used, the track modulus is identical to the direct fixation track analysis indicated in Article 4.4.2.2. Except in very special applications/installations, such track designs are generally no longer used in North America, largely due to the adoption of the relatively inexpensive and hence popular booted rail embedded track design. Details similar to open trough designs are seen in photographs of some European projects, but engineering details are not readily available. Embedded track designs of this sort are generally no longer recommended since the voids necessary for the direct fixation rail fasteners can collect moisture, leading to corrosion that can possibly compromise the structural and electrical integrity of the system.

Determining the track modulus for most embedded trackwork designs is more difficult than for direct fixation track for the following reasons:

- The rail is continuously supported. The Talbot premise of beam supports on an elastic foundation does not apply.
- Rail deflections can be extremely small.
- The spring rate for the rail support material is neither known nor easily determined.
- The subgrade stiffness, which is not well known, strongly affects the track stiffness.

Track modulus values have very little meaning for designs where the bare rail is completely encased in concrete without rail boots, such as occurs in some "bathtub" embedded track designs. Rail deflections, if any, are extremely small—possibly as low as 0.001 inches [0.025 millimeters]. The corresponding track modulus is extremely high and largely dependent on the deflection, if any, of the underlying subgrade. Extremely stiff track of this type is highly prone to corrugation and therefore not recommended.

An embedded track design with limited resiliency, such as a rail trough filled with a polyurethane/cork mixture, could have track deflection measurements under a 12,000-pound [53,379-N] wheel load in the range of 0.002 to 0.010 inches [0.050 to 0.25 millimeters]. The smaller deflection corresponds to an average force per unit deflection of the rail of approximately 2,000,000 lb/in [350,256N/mm]. The track support modulus is thus very high.

A more complex evaluation would be needed for a design that uses rigid, non-resilient, direct fixation rail fastener plate supports. For concrete infill, the track modulus would be extremely large. For an elastomeric or asphalt infill, the track modulus would be calculated from the rail deflection between rigid supports using conventional structural continuous beam formulas. However, the compliance of the base or subgrade would control the track stiffness.

The "rail boot" design, first employed in Toronto circa 1990, has become common in embedded track design. The boot provides a continuous elastomeric pad under the rail base, providing resiliency based on voids in the boot configuration, rail perimeter mechanical protection to the surrounding embedment materials, and electrical insulation to isolate the rail and prevent stray current leakage.

Representative track moduli for embedded track with rail boot may be estimated from data derived by one manufacturer. The manufacturer's rail boot design uses a 73 Durometer elastomer with a $^{5}/_{16}$-inch [7.9-millimeter] thickness under the rail base that has ribbed shape factors for resiliency. The static track modulus for this design varies, but is in the range of 15,000–30,000 lb/in^2 [103–207 N/mm^2]. An additional ribbed elastomer layer can be used under the boot, increasing pad thickness to ¾ inch [19 mm] and decreasing track modulus by approximately 50% to 65%.[3] Note that the track modulus change is not a linear function of elastomer thickness, but varies with the elastomer pad shape factor and use of a foamed elastomer.

Where the assumption of a linear elastomeric pad deflection is reasonable, a rough estimate of track modulus can be obtained by using a rail deflection of 15% of the elastomer pad thickness.[4] Elastomers that are routinely strained more than about 25% of their thickness begin to creep and attain a permanent set.

4.4.3 Transition Zone Track Modulus

Track modulus can vary dramatically among various track types. Well-maintained ballasted track in embankment soil of optimum density, where timber or concrete cross ties are supported by a stipulated depth of ballast and subballast, can have a track modulus as low as 2,500 psi [17.2 N/mm^2] or as high as 7,000 psi [48.3 N/mm^2]. Concrete cross tie and timber cross tie track with elastic rail fastenings tend toward the higher end of the scale. Embedded or direct fixation track, where a concrete base slab supports the rail, typically have a higher modulus value and greater

stability, as do non-ballasted "open" deck bridge structures where the rail is supported on rigid structural abutments and spans.

4.4.3.1 Interface between Track Types

The transition interface points between embedded and ballasted track segments and between direct fixation and ballasted track are typically locations of sudden major changes in track modulus. These differential track modulus values, if substantial (greater than 3,000 psi [20.7 N/mm^2] difference between trackforms), generate a weak spot in the overall track structure leading to high maintenance and likely breakdown of the track. If special design consideration is not given to such areas, particularly in line segments where the transit vehicles operate at speeds greater than typical yard operation, the ballasted track will invariably settle and the stiffer adjacent track installation may incur track component and structural damage. Every at-grade railway/roadway crossing also experiences the same track modulus changes. The passengers will experience degraded ride quality as an abrupt transition in the form of vertical acceleration, similar to an automobile hitting a pothole or bump in a highway. The abnormal ride quality is more pervasive when traveling from stiff track (high modulus) to the more flexible track (low modulus) than it is in the other direction.

A typical example is the interface between an open deck bridge and adjoining ballasted track. Railroads have long been aware of track alignment problems in these areas and have attempted to compensate by installing transition or approach ties similar to those shown on AREMA Plan basic Number 913. Various arrangements of long-tie installations are used on different railroads, sometimes with an incremental decrease in the cross tie spacing. The objective of these designs is to gradually stiffen the ballasted track structure over an extended distance, thereby reducing the abrupt change in track stiffness at the bridge abutment. Transition tie arrangements have also been placed at the ends of concrete tie installations where the track modulus differential between the concrete and timber cross ties often results in additional surface maintenance requirements. Similar conditions exist in transit track design where installations between ballasted track and both embedded and direct fixation track cannot be avoided. Special transition track design must be considered to maintain an acceptable ride quality at these locations without incurring excessive maintenance costs.

TCRP Research Results Digest 79: Design of Track Transitions, which reports results from TCRP Project D-7, Task 15, includes extensive information on track transition areas, both from actual installations and theoretical analysis, along with design guidelines.

4.4.3.2 Transition Zone Track Design Details

In North America, the usual design to compensate for the track modulus differential is to use a reinforced concrete transition slab (also commonly called an "approach slab") to support the ballasted track. These transition slabs (see **Figure 4.4.1**) extend from the end of the structure abutment or the end of embedded track slab, a minimum of 20 feet [6.1 meters] into the ballasted track section. The top of the slab typically is located 12 inches [300 millimeters] below the bottom of the ties immediately adjacent to the stiffer track, gradually increasing to 14 inches [355.6 millimeters] at the far end of the slab. This design replaces compressible subballast materials with a stiffer base, while also gradually decreasing the thickness and compressibility of the ballast layer.

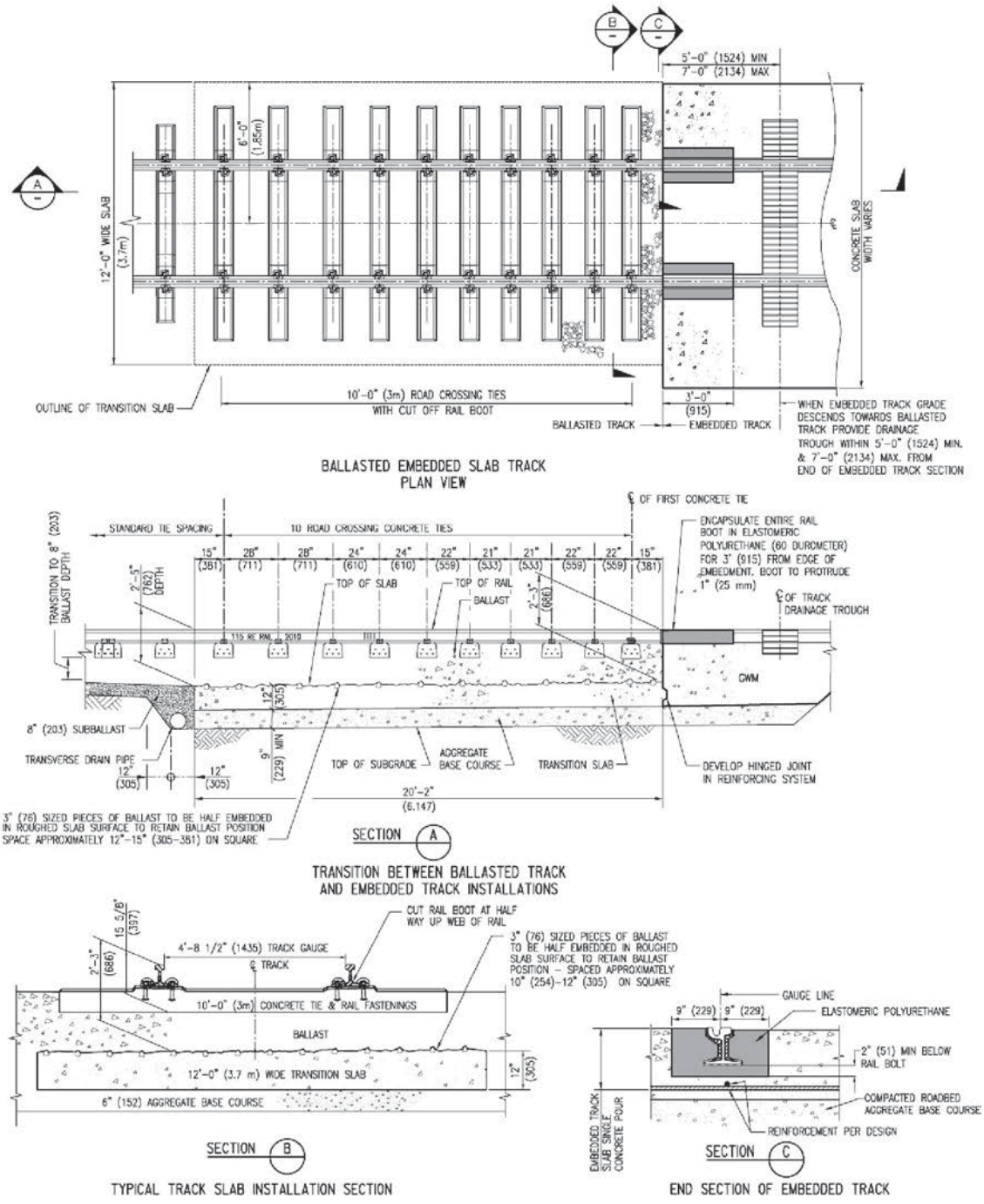


Figure 4.4.1 Track transition slab

Center-to-center distances between cross ties are generally reduced in the transition slab section to provide additional stability and increase the track modulus. Cross tie lengths are also often increased incrementally for the same reason, and such arrangements have been standard details for most freight and passenger railroads and many transit agencies for a century or longer. Curiously, computer simulations conducted by TCRP Project D-7, Task 15, concluded that such

measures had little benefit in terms of either reducing rail deflection or increasing track stiffness.[6] Additional research might be warranted into this topic.

Even a well-designed transition zone will experience some track surface degradation during operation, requiring periodic inspection and resurfacing to avoid pumping track conditions. Drainage conditions and design have a key role in establishing a high-performance transition zone. If the surrounding ballasted roadbed at the transition slab is well drained, the propensity for settlement will be reduced. On one project, with a transition zone that is always dry because it is underneath a building constructed over the trackway, 25 years of operation has resulted in no discernible settlement.

4.4.3.3 Transition Zone Conditions

The vertical deflection of the rail with a transition zone resembles a sine curve produced by the wheel load both entering and leaving the stiffer track section. The rails in the ballasted track portion will ultimately show a downward deflection approximately 3 feet [1 meter] from the transition point or end of direct fixation or embedded concrete slab, with a resulting upward force of approximately 3 feet [1 meter] into the direct fixation or embedded track portion. This, by itself, is not an issue if both sides of the transition are ballasted track, as would occur at the abutment of a ballasted track bridge. However, it is a concern where the stiffer side of the interface is either direct fixation or embedded track. The rail sine wave merely disturbs the ballasted track but attacks the direct fixation or embedment track installations with higher vertical loadings, leading to deterioration of components and track conditions.

4.4.3.3.1 Transition from Ballasted Track to Direct Fixation Track

The ballasted track side of the transition zone, even with a transition slab, cannot consistently produce a uniformly varying track modulus due to the tendency of ballast to compact, pulverize, and become fouled. Such deterioration leads to settlement voids, hard spots, and pumping track. Regular maintenance of the ballast is needed to protect the track structure's components and maintain ride quality.

Direct fixation fastener design continues to evolve, and a wide range of fastener spring rates is available. A direct fixation track modulus of 3,333 psi [23.1 N/mm^2], which compares favorably with conventional concrete cross tie installation, is now possible. Softer direct fixation fasteners in the zone immediately adjacent to the ballasted track transition zone can alleviate some of the transition problems that are not addressed by conventional transition slabs. Gradually increasing the spacing of the transit system's standard direct fixation fasteners on the approach to the ballasted track limit might also reduce the abrupt change in track stiffness at the interface without adding a special design of direct fixation fastener into the track maintenance inventory.

4.4.3.3.2 Transition from Ballasted Track to Embedded Track

Embedded track design continues to evolve and improve; however, the rail deflections that would be required to match typical ballasted track modulus values are difficult to achieve in embedded track. The track sine wave phenomenon in the rail places extremely high bending forces in the rail contained within the embedded track zone immediately adjacent to the ballasted-to-embedded track transition point. The differential in track modulus between ballasted and embedded track may be too large to overcome by introducing a flexible rail support in only the ballasted area adjacent to the interface. Introduction of additional resiliency in the embedded

track in advance of the interface is suggested. In the case of embedded track using rail boot, this might require placing additional elastomeric pads beneath the boot or transitioning to a trough type of embedded track with additional polyurethane grout beneath the base of rail. Keep in mind that elastomers provide resiliency only if they have some void into which they can bulge.

Transition areas that are operated at slow speeds (such as those that occur at the edge of a shop building apron) typically don't require any special treatments. As speeds increase, more thought should be given to gradually reducing the track stiffness over a time interval of a second or more. This may require measures on both sides of the interface location.

4.4.3.3.3 Design Recommendation

The goal of any design to improve the performance of the transition track zone is to minimize dynamic loads by equalizing or smoothing the vertical support condition and the dissipation of dynamic energy across the transition. The track designer must eliminate, reduce, or accommodate the pronounced sine curve reaction in the rail through the transition zone. Eliminating or reducing the sine curve using conventional track components is more easily achieved in direct fixation track than in embedded track. A recommended reading on transition zones is *TCRP Research Results Digest 79*. It reviews and analyzes various track transitions and designs among ballasted and non-ballasted track forms and structures and offers guidance to improve track and operating performance. *TCRP Research Results Digest 79* says the following transition designs can be considered the most efficient for rail transit applications, based on a literature review and GEOTRACK analysis:

- Matching the vertical fastener stiffness of direct, ballasted deck, or open deck bridges to the track modulus and rail deflection behavior of the at-grade ballasted track, without modifications of the at-grade track, provides the most efficient and cost-effective design. Direct fixation fasteners with stiffness values between 100 and 200 kip/inch [17,500 and 35,000 N/mm] deflection, are compatible with ballasted tracks with average stiffness subgrades (Er values between 5 and 15 ksi [34.5 and 103.4 megapascals]). The analysis showed the rail deflection differentials for these designs to be less than 0.04 inches [1 millimeter] for wheel loads of 12, 15, and 22.5 kips [5,443, 6,804, and 10,206 kg force respectively].

- The use of a rubber pad, bonded to the bottom of the concrete ties on ballasted deck bridges, provides adequate resilience to transition to ballasted track on an average stiffness subgrade. Modeling suggests that the rubber pad stiffness should be 100,000 lb/in or higher.

- Low stiffness subgrades with Er values less than 5 ksi [34.5 megapascals] require some modification in addition to the controlled resilience of the structure track. These subgrades are typically made up of cohesive soils (clays and silts) with moisture contents higher than optimum. Increasing the modulus of track on a low stiffness subgrade requires modification of the physical state of the soil and/or installation of a structural reinforced layer between the ballast and subgrade, such as a concrete approach slab.

- Avoid the creation of weak subgrade conditions during new construction by careful soils selection and the application of geotechnical best practices.

Additional suggested design features include the following:

- Diverting surface runoff from the direct fixation track or embedded track sections so that it doesn't enter the transition area. In direct fixation track, provide an end barrier wall and drain surface runoff to the side of the track beyond the embankment. In embedded track, provide a surface drain within 5 feet [1.5 meters] of the end face of the embedded track.

- Using a series of progressively longer concrete ties leading up to the abutment or embedment face of the non-ballasted track. Additional abutment width should be provided to accommodate a wider concrete base track slab and a wider embankment section to retain the widened track structure.

- Providing lateral perforated track drains at the ends of the base slabs to carry off base slab runoff.

- In embedded track, encasing the last 2 feet [60 cm] of booted rail prior to the beginning of the ballasted track in 60 durometer polyurethane. This will provide a track stiffness transition and protect the rail and pavement against damage that could occur when mechanically raising and tamping the adjoining ballasted track. The use of porous filler materials, such as cork of shredded rubber, can enhance the resiliency.

4.5 BALLASTED TRACK

Ballasted track is the most prevalent track type used in light rail transit. While ballasted track for light rail transit resembles conventional railroad track in appearance, its design may have to contend with issues such as electrical isolation and acoustic attenuation. In addition, ballasted LRT track may include continuous welded rail on an alignment that includes curves far sharper and grades far steeper than would ever be encountered on a freight railroad or even a "heavy rail" transit route.

Proper design of the roadbed, ballast, and subballast elements of the track structure is a key issue. It is essential in providing an adequate foundation for the track so as to minimize future maintenance requirements. Roadbed and ballast sections should be designed to minimize the overall width of the right-of-way while providing a uniform and well-drained ballast foundation for the track structure.

4.5.1 Ballasted Track Defined

Ballasted track can be described as a track structure consisting of rail, tie plates or fastenings, cross ties, and the ballast/subballast bed supported on a prepared subgrade. The subgrade may be a compacted embankment or fill section, an excavation or cut section, a bridge structure, or a subway tunnel invert. Ballasted track is generally the standard for light rail transit routes that are constructed on an exclusive right-of-way.

Ballasted track can be constructed to various designs, depending on the specific requirements of the transit system. Depending on the portion of the system under design and presuming for the moment that stray traction power currents are not an issue, a satisfactory ballasted track design

could consist of either timber cross ties with conventional tie plates, cut spikes, and rail anchors or concrete cross ties with elastic rail fastenings that incorporate conventional insulating components (so as to retain traction power currents within the rail). While the loadings typically are limited to those of the light rail vehicles only, heavier loading standards may be required. The track designer must consider that the heaviest loading may be generated by the maintenance-of-way equipment. In addition, ballasted track may need to accommodate freight railroad loadings where the track is to be shared with a commercial railroad. Light rail track structural loading is one-quarter to one-third of that imposed on freight railroad tracks. (Light rail bridges and aerial structures must also take these design parameters into consideration. Refer to Chapter 7 for structural design details.)

4.5.2 Ballasted Track Criteria

To develop ballasted track design, the following track components and standards must be specified:

- Rail section.
- Track gauge.
- Guarding of curved track and restraining rail features.
- Rail fastenings and tie plates.
- Type of track cross ties and corresponding track structure to suit operations.

4.5.2.1 Ballasted Track Rail Section and Track Gauge

Refer to Article 4.2 and Chapter 5 of this Handbook for guidance on determining rail section, track gauge, and flangeway requirements.

4.5.2.2 Ballasted Track with Restraining Rail

Refer to Article 4.3 herein for determining requirements, locations, and limits for guarding track with restraining rail. Specific details for various types of restraining rail designs are included in Chapter 5.

4.5.2.3 Ballasted Track Fastening

Refer to Chapter 5 for requirements concerning cross tie rail fastenings. A key issue for rail fastenings on ballasted track cross ties for transit use is providing sufficient electrical isolation to deter the migration of stray traction power currents.

4.5.3 Ballasted Track Structure Types

There are generally two standard designs for track structures on ballasted track:

- Timber cross tie track.
- Concrete cross tie track.

Both plastic and steel cross ties have been used in railway track construction, but they have not gained wide acceptance. See Chapter 5 for additional discussion on alternative cross tie materials.

Many transit systems have used both timber and concrete cross ties. Up until about 2000, the main line tracks on most new LRT installations were usually constructed using concrete cross ties

with standard rail insulation. The yard maintenance facility tracks were generally built with timber cross ties either with or without insulated fasteners. The non-insulated construction was appreciably cheaper to construct. Special trackwork in both main track and yard track was commonly constructed on timber switch ties, largely because concrete switch tie designs had not matured and were hence extremely expensive.

With very few exceptions, projects since about 2000 have mostly used concrete cross ties throughout, including yard tracks and special trackwork. This is largely because the cost of concrete ties in relation to high-quality timber ties with insulated rail fastenings is now comparable. Improved designs also show more promise for actually fulfilling the 50-year service life long claimed for concrete ties. By contrast, LRT systems constructed with timber typically face a need to replace a huge percentage of their cross ties during a fairly brief period—about 20 to 30 years after original construction. Also, transit yard design has been trending toward full electrical isolation of yard tracks from ground, separate traction power substations notwithstanding. Whether this is fully justified is an open question.

Ballasted track design can result in a suitable track structure using either timber or concrete cross ties. The differential track support or track modulus dictates the quality of the track, the ride, and future maintenance requirements. Concrete cross tie ballasted track provides a more reliable track gauge system and tighter gauge construction tolerances. The higher track modulus results in a smoother ride with less differential track settlement.

Chapter 2 documents the types and magnitudes of loads transferred from the vehicle wheel to the rail. The rail must support the vehicle and the resulting loads by absorbing some of the impact and shock and transferring some forces back into the vehicle via the wheels. The initial impact absorber on the vehicle is the elastomer in the resilient wheels (if used) followed by the primary suspension springs and then the secondary suspension system. The initial impact absorber on the track is the rail, particularly the rail head, followed by the fastening or supporting system at the rail base, and then the remaining track structure. A resilient rail seat pad is used to absorb some of the force on concrete cross ties. On timber cross ties, the resiliency in the wood itself acts as the absorber. All components absorb and distribute a portion of the load.

The track structure's design (degree of resiliency) dictates the amount of load distributed to the rail and track structure and the magnitude of force returned to the wheels and vehicle.

4.5.3.1 Ballasted Track Resilience

Ballasted track design allows partially controlled rail deflection in both the vertical and horizontal directions. This phenomenon of rail action contributes to successful track operation by distributing the load to the surrounding track components and structure.

Specific track design decisions must be made regarding the type of track structure (timber cross tie/concrete cross tie) and corresponding track structure resiliency or track support stiffness.

Rail supported on timber cross ties and a moderate ballast/subballast section using conventional rail fastenings consisting of tie plates, cut spikes, and rail anchors results in a track modulus range of 2000 to 2500 lb/inch per inch of rail [14 to 17 N/mm^2].

Resilient rail base pads are placed on concrete cross ties to protect the concrete tie seat and to impede the impact and vibration associated with wheel passage from migrating from the rail to the cross tie. Resilient rail base pads are a determining parameter of track modulus. A reduced pad height of 1/4 inch [6 millimeters] and a very stiff elastomer or polyethylene pad produce a stiff track support resulting in an increased rail support modulus.

Rail supported on concrete cross ties and an ample ballast/subballast section has a track modulus range of 4,500 to 6,500 lb/inch per inch of rail [31 to 45 N/mm^2].

4.5.3.2 Timber Cross Tie Ballasted Track

On many light rail transit systems, particularly legacy systems and systems constructed in the early 1980s, timber cross ties were considered to provide sufficient electrical isolation. Specific measures to insulate the track were not used because other measures were either taken or already in place (such as utility bonding and drain cables) to address traction power stray current. Typically, non-insulated rail fastenings were employed only in yard tracks, where the yard has its own traction power substation and stray currents are unlikely to leave the immediate site. Non-insulated, ballasted track was also occasionally used in rights-of-way where there were no parallel utilities; however, the occurrence of rights-of-way without parallel utilities is an extremely unlikely circumstance and the practice of using non-insulated track in such a situation ignores the fact that stray currents can take very circuitous paths quite distant from the track. Non-insulated track is therefore not recommended, and contemporary designs typically incorporate insulation systems within the cross tie rail fastening to control stray currents close to their source.

Timber cross tie ballasted track consists of the rail placed on a tie plate or rail fastening system that is positioned on the cross tie, which is supported by a ballast and subballast trackbed. Timber cross tie ballasted track is generally similar to the concrete cross tie track shown in **Figures 4.5.1** and **4.5.2**.

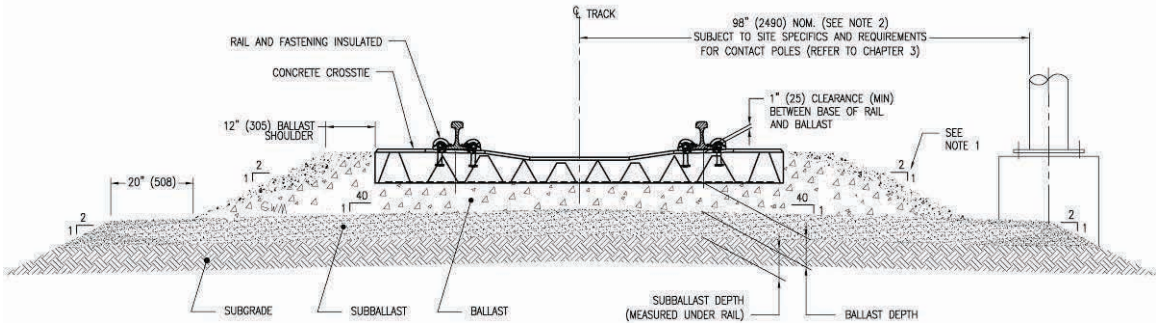


Figure 4.5.1 Ballasted single track, tangent track (concrete cross ties)

4.5.3.2.1 Timber Cross Tie Rail Fastenings

Conventional tie plates, cut spikes, and rail anchors were considered sufficient for ballasted track installations using timber cross ties for railroad and legacy rail transit track. However, current transit track design generally includes insulation in the rail fastening system so as to protect the negative return rail from stray electrical currents.

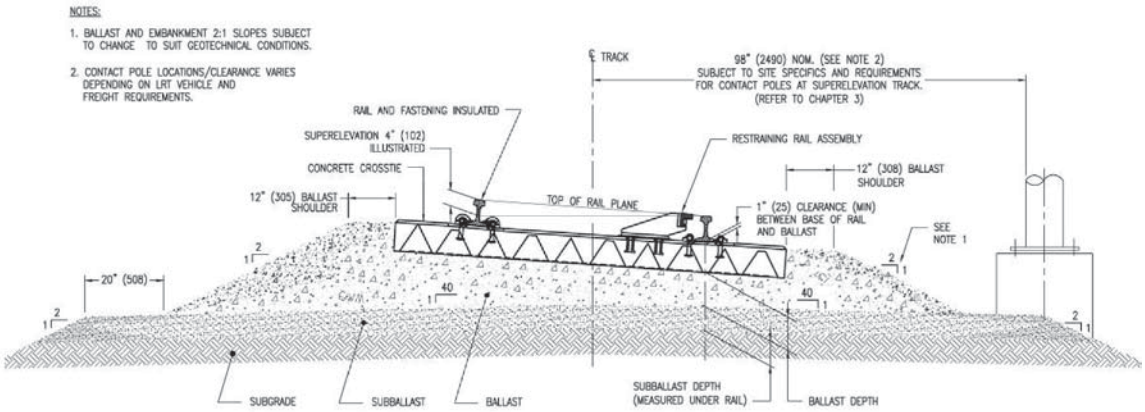


Figure 4.5.2 Ballasted single guarded curve track (concrete cross ties)

Although wood is an insulating material, timber cross ties provide only a limited barrier against stray current and become less effective in that regard over time. Therefore, timber cross ties generally utilize rail fastenings that are insulated at the base of the tie plate or fastening plate. A typical detail places a high-density polyethylene (HDPE) pad, ⅜-inch [9-millimeters] thick, between the timber cross tie and the tie plate. The HDPE pad will project a minimum of ½ inch [12 millimeters] beyond all sides of the steel fastening plate so as to minimize the chance of the edges being bridged by conductive debris. A special insulating collar/thimble is positioned in the anchor screw spike hole to isolate the screw spike from the steel fastening plate. The screw spikes are sometimes epoxy coated for additional electrical isolation. Alternatively, the hole drilled in the cross tie can be partially filled with a coat tar epoxy or other insulating gel prior to installing the spike, thereby forcing the insulating material into as many crevices and voids as possible. For additional design information on timber cross tie fastenings, refer to Chapter 5.

4.5.3.2.2 Timber Cross Ties

Timber cross ties have been standard for light rail transit installations for years and continue to be the standard for older, established transit agencies. Life cycle cost comparison of timber ties and concrete ties must be performed using a uniform baseline, including all fastenings and hardware needed for each type of tie. The tie spacing for timber ties is generally shorter than for concrete ties, which results in not only more cross ties, but also less ballast per unit of track length. These considerations must be factored into the analysis. Conventional rail anchors projecting into the ballast section will create a stray current leakage path, particularly in areas where the ballast is wet and/or contaminated, which is another issue to be considered. Also, the material cost for timber cross ties can vary widely over a short period of time. That said, many transit agencies still continue to use timber ties with satisfactory results. Broad gauge LRT systems (all of which are legacy operations dating back to the 19th century) generally select timber cross ties. It is unclear whether the deciding issue is first cost of special design concrete ties or a disinterest in change.

Timber cross ties (if selected) for a transit system should be hardwood (e.g., oak, maple, or birch), generally with a cross section of 7 by 9 inches [175 by 230 millimeters]. In the western

portions of North America, Douglas fir is readily available and considered equivalent to eastern hardwoods.

For additional information on timber cross ties, refer to Chapter 5. Determining timber cross tie spacing for transit track is discussed in Article 4.5.4.

4.5.3.3 Concrete Cross Tie Ballasted Track

Concrete cross ties have become nearly universal for new light rail transit installations. They have been shown to have a longer service life, have lower life cycle costs, provide a higher track modulus (which equates to better ride quality), and incur lower track surfacing maintenance costs. When the cost of procuring and installing insulated rail fastenings on high-quality timber cross ties is considered, concrete cross ties have a very favorable first cost, particularly considering that they can generally be spaced more widely than timber ties. The only exceptions in recent times have been extensions or rehabilitation projects on existing systems that have traditionally used timber cross ties. In some instances, those systems also use broad track gauge, which may have tipped first cost economics in favor of insulated timber versus concrete cross ties.

The concrete cross tie is typically insulated at the base of the running rail, thereby protecting the base of the rail from potential stray current leakage. Concrete cross tie ballasted track consists of the rail placed in the rail seat area and the tie supported by a ballast and subballast trackbed, as shown in **Figures 4.5.3** and **4.5.4**.

4.5.3.3.1 Concrete Cross Tie Rail Fastenings

Experimental concrete cross tie designs first appeared around 1920, but they were generally unsuccessful, largely due to failures in the rail fastening systems. The current success of the concrete cross tie is partly due to the introduction of elastic (spring) clip fastenings at the rail hold down location, which replace the spikes and threaded fasteners used in early designs. Fastening designs have also evolved to meet new requirements for electrical isolation.

The insulating barrier must be at the base of the rail or mounting surface to provide electrical isolation of the rail from the surrounding track components. The insulating barrier consists of a base rail pad and clip insulators for the edges of the rail base. As shown in Chapter 5, Figure 5.4.1 of this Handbook, the rail is fully insulated from the mounting surface.

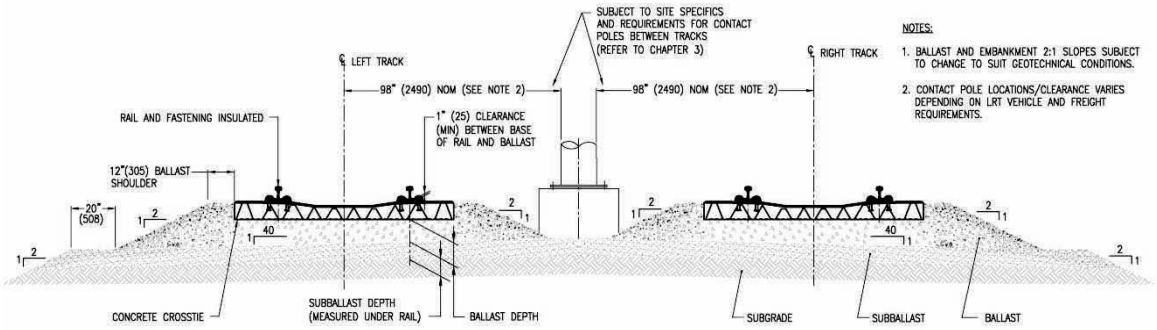


Figure 4.5.3 Ballasted double tangent track (concrete cross ties)

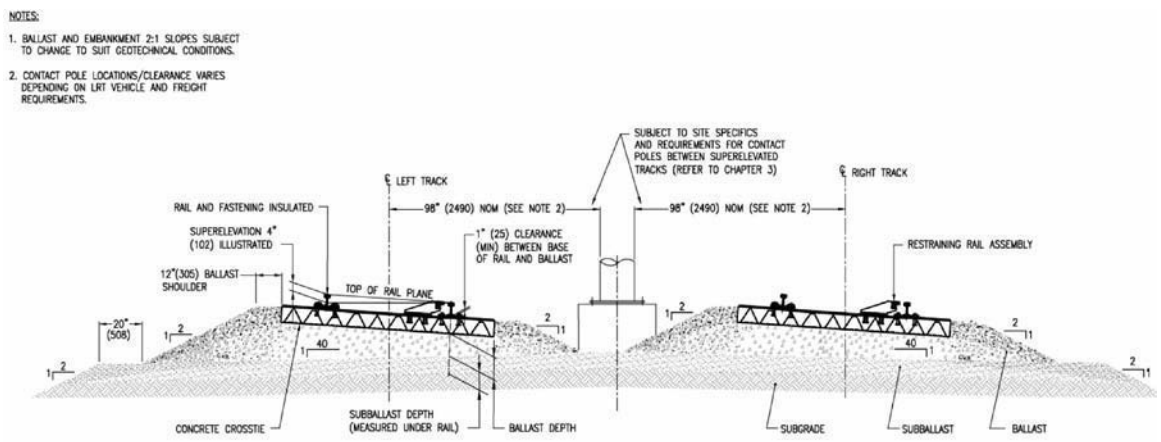


Figure 4.5.4 Ballasted double curved track (concrete cross ties)

The concrete cross tie design includes the specific type of elastic fastening system (e.g., spring clip) with insulating rail seat pad and rail base clip insulators. The two elastic clips at each rail seat provide sufficient toe load to the rail base to act as the longitudinal rail anchor, eliminating the conventional rail anchors used with timber cross ties.

4.5.3.3.2 Concrete Cross Ties

The typical transit concrete cross tie is made of prestressed, precast concrete produced in a factory with climate controls for the curing process. The ties are generally 10 inches [255 millimeters] wide and 8' 3" [2515 millimeters] long, measured at the base of tie. So as to facilitate removal from the molds, the tie is vertically tapered, with slightly smaller plan dimensions at the top of the tie. Tie thickness is generally 7 ½ inches [190 millimeters] at the rail seat and 6 ½ inches [165 millimeters] at the center of the tie. For additional information on concrete cross ties refer to Chapter 5.

4.5.4 Cross Tie Spacing

The optimal spacing of cross ties in ballasted track is dependent on two issues:

- Vertical support, so as to distribute the wheel loads through the ballast and subballast such that the underlying soils are not overstressed.

- Lateral support, so that the track is adequately restrained against lateral movement due to thermal stresses and loadings in the rails.

4.5.4.1 Cross Tie Spacing—Vertical Support Considerations

Ballasted track structure design is dependent on the vehicle wheel load, a predetermined track modulus target or standard, the selected rail section, the type and size of tie, and the depths of ballast and subballast. These are combined to meet the criteria established by AREMA for both ballast pressure and subgrade pressure.

Ballasted track designs can meet or exceed the AREMA pressure requirements by altering the variable parameters (track modulus, tie spacing, and ballast depth) as needed. As a guideline, the following sample calculations—based on the formulae from Talbot[5], Timoshenko and

Langer[11], and Hay[2]—are provided for design of ballasted track with timber or concrete cross ties assuming the following typical LRT installation parameters:

Rail Section	115 RE
Vehicle Load per Wheel	12,000 pounds [5,400 kilograms]
Track Modulus	
Timber Tie	2,500 lb/inch per inch [17.2 N/mm^2]
Concrete Tie	5,000 lb/inch per inch [34.5 N/mm^2]
Desired Load Transfer to	
Ballast	<65 psi [0.45 MPa]
Subgrade	<20 psi [0.14 MPa]
Ballast Depth	10 inches [255 millimeters]
Subballast Depth	8 inches [200 millimeters]
Tie Sizes	
Timber	7 x 9 x 102 inches [180 x 230 x 2590 millimeters]
Concrete	7.5 x 10 x 99 inches [190 x 250 x 2515 millimeters]

Design Calculations:

Tie Seat Load = $\beta \times a \times P$ (Timoshenko and Langer[11])

where

a = tie spacing (variable)

P = axle load = 107 kN (24 kips)

$$\beta = \left(\frac{u}{4EI} \right)^{1/4}$$

Timber Tie: u = track modulus = 2,500 lb/inch per inch [17.2 N/mm^2]

Concrete Tie: u = track modulus = 5,000 lb/inch per inch [34.5 N/mm^2]

E = modulus of rail steel = 30 x 10^6 psi [206,800 N/mm^2]

I = moment of inertia of 115 RE rail = 65.9 in^4 [27.4 x 10^6 mm^4]

Tie Bearing Area = tie width x tie length

Timber = 9 inches x 102 inches [230 x 2590] = 918 sq inches [595700 mm^2]

Concrete = 10 inches x 99 inches [250 x 2515] = 990 sq inches [628750 mm^2]

$$\text{Ballast Load} = \frac{\text{Tie Seat Load}}{2/3 \text{Tie Bearing Area}} \quad \text{Hay}^{[2]}$$

$$\text{Subballast Load at Tie Centerline} = 1.23 \frac{\left(\frac{\text{Seat Load}}{\text{Tie Bearing Area}} \right) \times \text{Tie Width}}{\text{Ballast Depth}} \quad \text{Talbot}^{[5]}$$

Subgrade Load at Tie Centerline is similar to subballast load calculation except depth includes ballast and subballast heights.

Using the above formulas, **Table 4.5.1** presents the values according to the parameters. Tie spacing can be determined from this table. Neither the AREMA recommended maximum ballast pressure, 65 psi [0.45 MPa], nor the maximum subgrade pressure, 20 psi [0.14 MPa], should be exceeded.

Table 4.5.1 Ballasted track design parameters

Track Modulus	Cross Tie Spacing inches [mm]	Tie Seat Load Kips [kN]	Tie-Ballast Load				Subballast Load		Subgrade Load Ballast + Subballast	
			9" [230] Tie		10" [250] Tie		10" [255] Ballast Depth		18" [455]	
			psi	[MPa]	psi	[MPa]	psi	[MPa]	[psi]	MPa
2500 lb/in/in [17.2 N/mm2]	20" [510]	11.4 [50.7]	18.5	0.127	n.a.	n.a.	13.7	0.094	7.6	0.096
β = 0.0237/in [0.00093/mm]	24" [610]	13.6 [60.7]	22.1	0.152	n.a.	n.a.	16.4	0.113	9.1	0.115
	27" [685]	15.3 [68.2]	24.9	0.171	n.a.	n.a.	18.5	0.127	10.3	0.130
	30" [760]	17.0 [75.6]	27.6	0.189	n.a.	n.a.	20.5	0.141	11.4	0.144
	32" [810]	18.1 [80.6]	29.4	0.202	n.a.	n.a.	21.8	0.150	12.1	0.153
5000 lb/in/in [34.5 N/mm^2]	20" [510]	13.5 [60.0]	n.a.	n.a.	20.4	0.142	16.8	0.115	9.3	0.115
β = 0.0282/in [0.0011 /mm]	24" [610]	16.1 [71.8]	n.a.	n.a.	24.3	0.170	20.0	0.138	11.1	0.138
	27" [685]	18.1 [80.6]	n.a.	n.a.	27.3	0.191	22.5	0.155	12.5	0.155
	30" [760]	20.1 [89.5]	n.a.	n.a.	30.3	0.212	25.0	0.172	13.9	0.172
	32" [810]	21.4 [95.3]	n.a.	n.a.	32.3	0.226	26.6	0.183	14.8	0.183

Note: 1 MPa = 1 N/mm^2

The preceding computations are representative of the calculations needed to design the ballasted track structure. The parameters that alter the actual design are predetermined track modulus, type of tie (timber or concrete), depth of ballast and subballast, and tie spacing. The challenge for the track designer is to combine these parameters to achieve the best life cycle costs and lowest maintenance costs.

4.5.4.2 Cross Tie Spacing—Lateral Stability Considerations

The above calculations determine the cross tie spacing and affect the track modulus or the vertical track stiffness. Lateral track stability can also affect cross tie spacing.

For the curve radii typically encountered in railroad work, if there are sufficient cross ties to provide vertical support, lateral restraint is rarely an issue. This is not true in LRT track design. The horizontal track alignment for a light rail transit system can include curves far more severe than curves on railway systems such as metro rapid transit, commuter rail, or freight railroads. Ballasted track alignment is far more difficult to construct and maintain in tight radius curves. Special consideration should therefore be given to increasing lateral track stability by reducing the cross tie spacing.

Lateral track stability is provided by ballast friction contact along the sides and bottom of the tie and by the end area of the tie. The end area of the tie provides a calculated degree of lateral stability; however, increasing the ballast shoulder width beyond an 18-inch [450-millimeter] limit provides no increase in stability. Reducing cross tie spacing, thereby increasing the number of ties, can increase lateral track stability. Timber cross ties have been proven to provide greater lateral stability than concrete ties, generally because the ballast's sharp edges penetrate the tie surfaces, increasing the friction and locking the cross tie in position. On the other hand, the concrete tie's increased weight also provides increased lateral stability. To improve the lateral stability of concrete cross ties, some tie manufacturers have developed a serrated or "scalloped" side tie surface, increasing the ballast's locking capabilities.

As a guideline, the track designer should consider reducing the conventional cross tie spacing calculated in the previous article by 3 inches [75 millimeters] for curves with radii less than 1000 feet [300 meters] and an additional 3 inches for curves tighter than 500 feet [150 meters].

To improve lateral stability, especially with conventional smooth-sided concrete ties, a tie anchor can be bolted to the tie. The tie anchor is a vertical blade penetrating below the base of the tie into the ballast bed. Tie anchors can be attached to all or alternate ties in the curve. Installation of tie anchors is a manual process that disturbs the ballast consolidation, requiring the track to be retamped. These devices appreciably complicate track construction and should be considered only as a last resort.

4.5.5 Special Trackwork Switch Ties

Concrete switch ties have been developed by the railroad industry to reduce installation costs and long-term maintenance on heavy haul freight lines. Concrete switch ties are initially expensive to design and fabricate, but now that some standard designs exist, procurement costs are coming down compared to pricing during the 1990s.

As of this writing (2010), most transit agencies still use standard timber hardwood ties for special trackwork for both main line and maintenance facility and storage yard installations. Concrete switch ties are becoming more popular, but have not become universally accepted, even on projects that otherwise use concrete cross ties. The situation is evolving, and it seems reasonably certain that concrete will become the switch tie material of choice for most LRT projects in the near term.

Turnout standards vary among transit agencies. Therefore, various concrete tie geometric layouts and designs would be required to meet the requirements of each agency. Standardization and simplicity in turnout tie design is advancing and is required to allow the transit industry to develop a uniform, economical, standard concrete switch tie set for various turnout sizes.

4.5.5.1 Timber Switch Ties

The present standard for timber switch ties is domestic hardwoods. In the eastern United States, oak is the preferred species for switch ties. In the western United States, Douglas fir is the predominant species for both switch ties and cross ties.

Tropical hardwood ties, manufactured from species such as Bonzai, Ekki, and Azobe have been used in North American railway and transit trackage with mixed success. The reader is cautioned about using these tropical woods. Thorough research on the specific wood of interest and the origin of the wood is recommended before a procurement is undertaken. Use of the correct botanical names is critical. Several species of Azobe wood exist, each with significantly different characteristics, and the inferior species are subject to rapid decay. Obtaining the correct material cannot be guaranteed without continuous on-site inspection at the saw mill as the species are nearly impossible to differentiate after the bark has been removed. See Chapter 5 for additional information on tropical hardwoods.

In North America, the typical timber switch tie is generally a 7 x 9 inch [180 x 230 millimeter] section in various lengths ranging from 9 to 17 feet [2,750 to 5,182 millimeters]. Overseas, timber switch ties are typically much thinner and somewhat wider. Such ties are not recommended for North American use.

Extra long timber switch ties, 22 feet [6,710 millimeters] and longer, may be required to accommodate special trackwork locations, such as crossovers and double crossovers where the track centers remain at a standard width. Alternate long-tie designs exist where two shorter ties are abutted and spliced together by a hinged connection. This design allows one track to be removed or worked on while the other track remains in service. The abutted tie connections can sometimes alternate in location (within the track gauge areas) between the two tracks, improving the installation's stability. The same articulated configuration can also be used with concrete crossover ties.

Similar to main line timber cross ties, timber switch ties may require an insulated switch plate design to protect against stray current leakage. Generally, insulation details for switch and frog plates are similar to those used on main line timber cross ties. The dual concern for both stray current control and vibration isolation has occasionally resulted in the installation of special trackwork direct fixation fasteners on timber switch ties.

4.5.5.2 Concrete Switch Ties

Concrete switch ties for virtually all size turnouts are now available for a price. However, the design details are generally not published information and readily available as they are proprietary to each manufacturer.

Concrete switch tie designs and layouts are different from the timber switch tie arrangements. Tie spacings are increased as the width of the concrete switch ties, approximately 10 inches [250 millimeters], distributes loads over a wider ballast surface area than timber switch ties. For simplicity and because patterns are readily available, spacings of ties beneath switches and frogs often follow railroad practice.

The lengths of the concrete switch ties conform to the needs of the special trackwork layout. Unlike timber switch ties, which usually are supplied in length increments of 1 foot [30 cm], concrete switch ties are often supplied in specific lengths for each tie location. The switch tie design includes embedments for mounting rails and fastenings for special trackwork plates. Both embedded shoulders and single rail fastener plates have been used outside of the areas of switch and frog plates.

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Chapter 5—Track Components and Materials

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CHAPTER 5—TRACK COMPONENTS AND MATERIALS

5.1 INTRODUCTION

The track components that form the track structure generally include steel rails, a rail fastening system, and an underlying structure that provides overall stability and strength. The most familiar form of trackwork is ballasted track, where cross ties embedded in ballast rock provide the last function. However, light rail transit includes several other types of trackforms, each designed to meet the needs of a particular trackway condition, such as public streets, aerial structures, and subway tunnels. This chapter discusses all of these trackforms and the sundry components used in each, as well as elaborating on the various designs and requirements.

The information in this chapter pertains to light rail transit systems that use overhead contact system (OCS) electric traction power distribution, with the running rail providing the negative return path. In addition, the rails are often used as a component of the signal system. While this discussion is specific to LRT, many LRT track components are common with those used on heavy rail metro transit systems and freight, commuter, and intercity railway lines.

5.2 RAILS

5.2.1 Introduction

Rail is the most important—and most expensive—element of the track structure. It is the point of contact with the vehicle wheel, the structural beam supporting the vehicle load, and one location where noise is generated. Hundreds of different rail sections have been created since the first strip of iron was placed over a longitudinal timber beam nearly 200 years ago. Each new rail section has been developed to satisfy a particular combination of wheel/rail loading in a specific trackway environment.

5.2.1.1 Types of Rail for Light Rail Transit

Two types of rail are in common use on light rail transit: tee rails and groove rails. Tee rails, so called because they vaguely resemble an inverted upper case letter T, were first developed for use in ballasted track. When rails were placed in paved streets, groove rails (commonly called girder rails in North America) were developed to provide the needed flangeway.

5.2.1.2 Rail Lengths

When manufactured for use by a railroad, rails are naturally delivered on railroad freight cars and because of the evolution of railroad freight cars, rail lengths have increased over the years. In recent times, North American standards for rail lengths have increased from the 39-foot [11.8-meter] lengths that prevailed until about 1975, up to 78-, 80-, and 82-foot [23.8-, 24.4- and 25-meter] lengths. The actual length of these longer rails varies due to each manufacturer's equipment. Since about 2005, some North American and European rail mills can now produce tee rail as long as 400 feet [122 meters]. These extremely long rails were developed in response to freight railroads' interest in reducing the number of rail welds necessary to produce continuous strings of rail. However, extremely long rails require special handling methods and equipment so as to avoid damage and obviously cannot be routinely moved over the highway. Should

favorable conditions for delivery and handling of such lengths prevail, rail transit projects could consider procuring such longer rail sections.

5.2.1.3 Joining Rails

Bolted rail joints between contiguous rails have always been the weak link in the track. Welding of the individual rolled rail lengths (sometimes called "sticks") into continuous welded rail (CWR) (called "strings") is now customary to eliminate bolted rail joints, improve the performance of the rail in track, and provide a quieter track system. On an electrified railway and any railway using track-circuit-based signaling, welded rail has the advantage of providing a better path for electric current. Continuous welded rail has been made possible by the development of rail welding systems. The two most prevalent rail welding systems are electric flash butt welding and exothermic (also known as "thermite") welding. Both types of welding are discussed in Article 5.6.1 of this chapter.

Flash butt welded CWR is the recommended standard for transit trackwork. The only exceptions are locations, such as within special trackwork and very sharp curves using precurved rail, where the flash butt welding equipment cannot be clamped to the rail. Thermite welding is therefore used in such locations. Rarely, under specific or unusual conditions, bolted jointed rail may be more practical to suit specific site conditions and future maintenance procedures. However, wherever a segment of bolted rail is proposed immediately contiguous to a CWR string, an analysis of the ability of the rail anchoring system to resist thermal forces should be undertaken so that "pull-aparts" do not occur during cold weather.

5.2.1.4 Rail in Curves

While generally very stiff, rail can be surprisingly ductile, particularly as long strings of CWR, and it can be field curved down to fairly tight radii without any special equipment. In the case of 115 RE rail, CWR can usually be laid down to a 300-foot [about 90-meter] radius without difficulty. Below that threshold, it is common practice to precurve the rail so as to eliminate internal stresses that attempt to straighten the rail. For additional information about the precurving of both groove rail and tee rail, refer Articles 5.2.2.5 and 5.2.3.5 in this chapter.

5.2.1.5 Rail Handling

See Chapter 13 of this Handbook, Article 13.3, for discussions concerning the handling of rail and other trackwork materials during construction.

5.2.1.6 Rail/Wheel Interface Issues

Wheel/rail interface is one of the most important issues in the design of the wheel profile and the railhead section. Trackwork engineers on freight railroads have a difficult time maintaining an optimized rail/wheel interface because they must accommodate an extremely wide array of rolling stock and wheel maintenance conditions. The sheer number of freight cars in North America, which are owned by hundreds of business concerns, makes it virtually impossible to maintain freight car wheels to tight standards. By contrast, light rail transit systems usually have relatively small fleets, often of only one or two vehicle types, and do not have to contend with vehicles from other owners. These "captive" vehicle fleets provide the opportunity to custom design and maintain an optimal wheel/rail interface, with a single standard for wheel profile that is designed to match the types of rail used on the system.

Vehicle operational and ride performance is highly dependent on the primary and secondary suspension systems that allow the vehicle to traverse the track system and negotiate track curves. The manner in which the wheels and axles are incorporated into the vehicle truck, together with the wheel and rail profiles, control how well the vehicle truck steers in curves and at what speed truck hunting will commence on tangent track. On trucks with tapered wheels rigidly mounted on conventional solid axles, the contact zone between the wheel and rail will migrate to a position near the gauge corner on the high outside rail of curves to improve steering. The contact zone on the low rail is best located toward the field side of the rail head. These two distinct contact zone locations take advantage of the tapered wheel rolling radius differential so as to automatically provide axle steering due to the conical shape formed by the different rolling radii between the two wheels. This action does not occur on trucks equipped with either independently rotating wheels or non-tapered (also known as "cylindrical") wheel tread profiles.

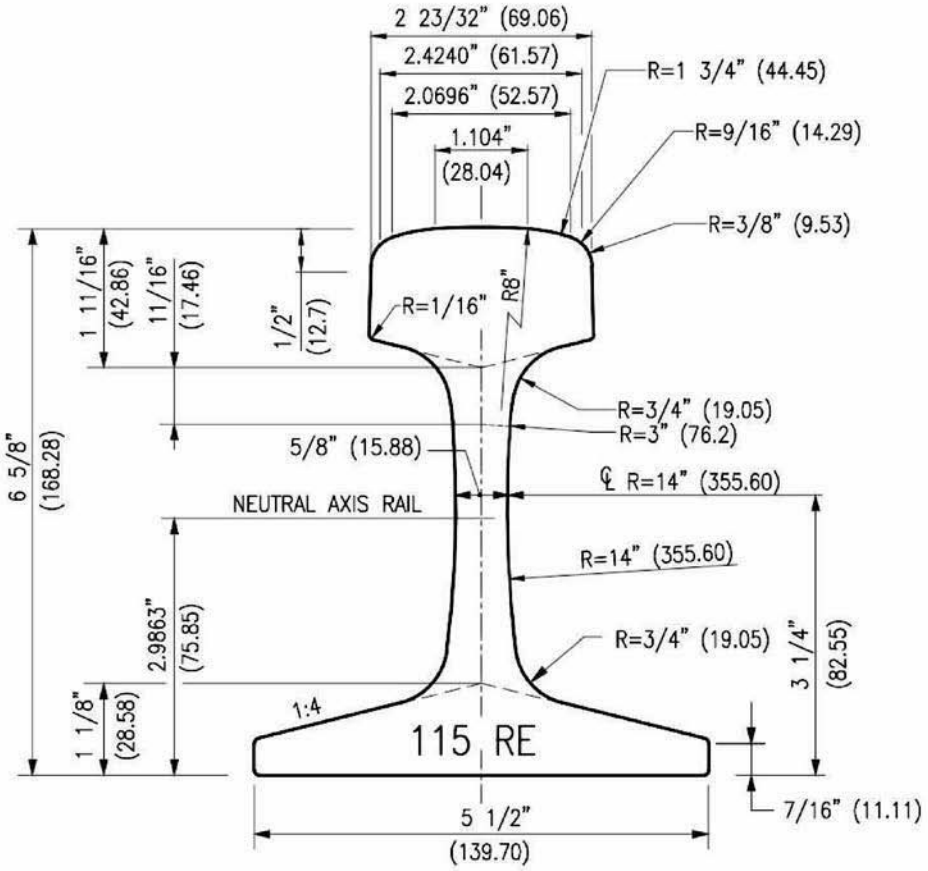
Wheel and rail design that produces a wider conformal contact zone, or a wider contact and wear pattern, will, after a short period of service life, result in poor vehicle tracking performance through curved track. A wider contact band can also exacerbate any tendency for truck hunting, which in turn is one cause of corrugations on the rail head. Conformal contact conditions are produced when the rail head radius is worn to a relatively flat condition and the wheel is worn to a similar flat or hollow condition. This stimulates rail head corrugation growth, producing an irregular wavy wear zone across the head of the rail. These corrugations result in unsatisfactory ride quality and excessive noise.

5.2.2 Tee Rail

Tee rail is the prevalent section for running rail on contemporary light rail systems for all three types of track structure (ballasted, direct fixation, and embedded). Overseas, tee rails are commonly referred to as either "Vignole rails" or "flat bottom rails," and dozens of sections are still manufactured. In North America, tee rail sections have evolved from about a hundred sections circa 1900 down to the American Railway Engineering and Maintenance-of-Way Association's (AREMA's) four standard sections—115 RE, 132 RE, 136 RE, and 141 RE. Several other rail sections (not documented here) are still manufactured for specific customers, generally in very small quantities and on an irregular schedule. Rail section designs, the composition of the steel rails are rolled from, and the post-rolling treatments used to increase their strength and resistance to wear all continue to evolve and be improved worldwide.

5.2.2.1 Rail Section—115 RE

Selection of the running rail section must be performed with consideration for economy, strength, and availability. The 115 RE rail section (see **Figure 5.2.1**) is the primary section used on current North American light rail track systems. This is largely because, as a recognized and popular standard section for freight railroad use, there is a guaranteed continuous supply from many manufacturers. The 115 RE section has more than adequate beam strength to support light rail vehicle wheel loads on standard spacings for cross ties and direct fixation fasteners and has sufficient cross-sectional area to provide a low-resistance negative return conductor in the traction power circuitry. As of 2011, because of the popularity, economics, and ready availability of 115 RE rail, there is virtually no reason to consider tee rails of non-North American standard designs.



115 RE RAIL	
RAIL AREA (SQ. IN.) HEAD WEB BASE WHOLE RAIL	 3.8861 3.0362 4.2947 11.2171
RAIL WEIGHT (LBS/YD) (BASED ON SPECIFIC GRAVITY OF RAIL STEEL = 7.84)	114.3757
MOMENT OF INERTIA ABOUT THE NEUTRAL AXIS	65.5
SECTION MODULUS OF THE HEAD SECTION MODULUS OF THE BASE	18.0 21.9
HEIGHT OF NEUTRAL AXIS ABOVE BASE	2.99
LATERAL MOMENT OF INERTIA	10.7
LATERAL SECTION MODULUS OF THE HEAD LATERAL SECTION MODULUS OF THE BASE	7.88 3.89

Figure 5.2.1 115 RE tee rail with 8-inch crown radius

The “115” and the “RE” in the rail section identification 115 RE mean the following:

- 115 is the mass (weight) in pounds per one yard length. It is rounded from the exact mass of 114.3757 pounds per yard and is equivalent to 56.737 kilograms per meter.
- RE = AREMA standard rail section.

From a structural perspective and quite likely from an electrical perspective, a rail weighing about 100 pounds per yard [roughly 49.6 kg/m] would be sufficient for rail transit. Notably, many European tramways and Stadtbahn operations use the CEN 49E1 section, which weighs very nearly 100 pounds per yard, for open track areas. The only 100-pound rail commonly available in the United States is “100-8,” a modification of the former 100 ARA-B section incorporating an 8-inch crown radius. It is rolled by only one manufacturer, mostly for the needs of one very large transit agency.

5.2.2.2 Rail Strength and Metallurgy

Chemical composition guidelines for the steel used to make running rail are standardized in the AREMA *Manual for Railway Engineering*, Chapter 4, for both standard rail and high-strength rail. The use of alloy rail is not recommended to obtain high-strength standards because of the additional complexities of welding alloy rail. The current AREMA standards for standard strength and high-strength rail hardness (developed by the head hardening procedure) are the following:

- Standard Rail—321 minimum Brinell Hardness Number (BHN).
- High-Strength Rail—341 to 388 BHN (may be exceeded provided a fully pearlitic microstructure is maintained).

The life of the rail can be extended [3] by increasing the rail’s resistance to

- Wear.
- Surface fatigue-damage.
- Fatigue defects.

Fatigue is rarely an issue in rail transit service since the loadings are much less than they are for railroad service and the plastic deformation that results from high contact stresses occurs much less often. Wear, on the other hand, is a significant issue in transit service, particularly in sharp curves. Rail steel hardness, cleanliness, and fracture toughness can increase resistance to wear. The effect of rail hardness in resisting gauge corner and gauge face wear is a known fact. Increased rail hardness in combination with minimized sulfide inclusions reduces the likelihood of rolling contact fatigue. This, in turn, reduces development of subsequent surface defects such as head checks, flaking, and shelly spots. Clean steel, free of oxide inclusions, combined with good fracture toughness, reduces the likelihood of deep-seated shell formations. Both shelly spots and deep-seated shells can initiate transverse defects, which ultimately cause broken rails.

Current rail standards include increased rail hardness and improved rail steel cleanliness, with the pearlitic steels peaking at 390 BHN. Research commencing in the 1990s focused on other metallurgies such as bainitic steels. Although bainitic steels of the same hardness as pearlitic steel are not as wear resistant, high-hardness low-carbon bainitic steel offers wear resistance superior to that of pearlitic steel.

A general guideline for transit installations is the use of clean rail steel with a hardness not less than

- 300–320 BHN (standard rail) in tangent tracks, except at station stops (and similar locations of heavy traction or braking) and gradients steeper than 4%.
- 380–390 BHN in tangent tracks at station stops, gradients steeper than 4%, curved track with radii less than 1640 feet [500 meters] and all special trackwork components including switch points, stock rails, guard rails, frog rails, and rails within the special trackwork area.

However, experiences during the 1990s and early 2000s suggest that there may be benefits to specifying head hardened rail steel in all primary tracks. Hardened rail is known to retard the growth of corrugations, reduce rail head flanging wear, and increase overall rail life. In addition, specifying a single rail type throughout a project simplifies both construction and future maintenance. Serious consideration should be given to this option for locations such as embedded track, where rail grinding with conventional equipment is difficult.

Directly related to rail chemistry is the matter of rail conductivity. So that the rail provides sufficient capacity for the negative return side of the traction power circuit, guidelines suggest that its electrical resistance should be a maximum of 0.0092 ohm/1000 ft at 68 degrees F [0.0302 ohm/km at 20 degrees C). Normal rail steel chemistry for any rail section likely to be used for LRT service meets this requirement. As a practical matter, any attempt to alter the rail chemistry so as to increase its conductivity (such as increasing the percentage of copper) would likely have adverse effects on the rail's mechanical properties. It is unlikely that commercial rolling mills would provide warranties for rail that has not been rolled to a recognized standard such as AREMA or an applicable European Norm.

5.2.2.3 Rail Straightness

An additional measure worth considering for locations where noise and vibration are particularly sensitive issues is the selective use of so-called "super straight" rail to improve ride quality and reduce noise by providing a more consistent contact band. For additional information on rail straightness and noise refer to Chapter 9, Article 9.3.3.7.

5.2.2.4 Rail Running Surface Finish

Running rails are rolled to specifications that have very tight tolerances on dimensions, including the profile of the rail head. Manufacturers generally have little difficulty meeting those tolerances when measured at the actual clean surface of the rolled steel. However, rail is generally delivered from the manufacturer with a bluish to black surface residue called "mill scale." Mill scale is composed of iron oxides, is typically about 1/32 inch [1 mm] thick, and is a byproduct of the hot rolling process. Mill scale is generally only loosely attached to the underlying steel, is usually discontinuous, and will frequently, albeit somewhat irregularly, flake off during handling of the rail from the steel mill to the track. So long as the mill scale is intact, it provides the underlying steel with an unintended but reasonably effective protective coating from corrosion.

Mill scale is of no real consequence on the base or web of the rail except that it must be removed from the vicinity of a rail weld before welding. In railroad service, any mill scale on the running surfaces of the rail is very quickly removed by abrasion due to the extremely high contact

stresses between the rail and the wheels. However, under transit wheel loadings, mill scale on the running surfaces of the rail will not wear away as quickly and can cause several problems:

- It can interfere with the reliable shunting of low-voltage signal circuits.
- It can interfere with traction power ground, causing arcing as the traction power current burns through the mill scale to find good ground on the underlying rail steel. This arcing can result in appreciable damage to the rail head and the vehicle wheels.
- The residual mill scale, together with any damage caused by arcing, can cause the contact band between the top of rail and the wheel to become irregular, resulting in degraded performance. This can initiate wheel dynamic responses that subsequently result in higher noise and, at worst, possible corrugation of the rail's running surface.

Because of these issues, it is customary for rail transit projects to lightly grind the running surface of newly laid rail so as to remove the mill scale and thereby provide a clean and uniform running surface for the wheels. The problem is that the grinding process itself can possibly damage the surface of the rail. Even under the best of circumstances, grinding alters the as-rolled geometry of the rail head and thereby possibly invalidates some of the assumptions made concerning how the wheel will interface with the rail. If the rail is in embedded track or has an adjoining restraining rail, the chances that grinding will alter the as-rolled rail profile into an undesirable contour are even higher.

These issues could be relieved if there was a reliable method of removing the mill scale on a production basis before the rail is laid in track and perhaps even at the time it is rolled. However, as of this writing, no such methods are commonly available for production use. Circa 1990, some success was achieved using the equivalent of a "belt sander," but the process was very slow, not well adapted to embedded track, and was thus never adopted as a standard practice. Blast cleaning methods might be promising, but it is not believed this has ever been attempted on any sort of production basis. The development of a production method for removing mill scale from the rail head, perhaps at the rolling mill, could result in a much better wheel/rail interface from the very start of rail operations. Additional research and development is very much needed in this area.

5.2.2.5 Precurving of Tee Rail

Where the track radius is sharp enough that springing the rail to radius and keeping it there would be difficult or perhaps even dangerous, the rail must be precurved. Precurving rail essentially requires stretching the rail beyond the elastic limit of the steel so that it cannot spring back to its original straight configuration. Precurving is sometimes desirable even when the radius is within the range where the rail can be sprung into alignment. Refer to discussions in Article 6.11 in Chapter 6 and Article 13.3.2.2 in Chapter 13.

These are the general guidelines for precurving 115 RE tee rail:

- Standard Rail
  - Precurve rail horizontally for curve radii below 300 feet (91 meters).
  - Precurve rail vertically for curve radii below 755 feet (230 meters).

- High-Strength Rail
  - Precurve rail horizontally for curve radii below 400 feet (120 meters).
  - Precurve rail vertically for curve radii below 980 feet (300 meters).

Rail should be precurved in the vertical plane whenever the mid-ordinate of the vertical geometry exceeds the natural sag or droop of the rail length being used. "Sag" means when the rail is supported only at the ends. "Droop" is when the rail is supported only in the middle. Vertical precurving can often be required when embedded track rails must conform to severely warped pavement surfaces in a street intersection.

Precurved rails are often needed in high-wear locations where the rail is replaced more frequently. These locations are sometimes designed with standard bolted rail joints rather than welded joints to facilitate rail change out. This does not necessarily work in practice since change out of individual rails in a worn sharp curve could result in significant gauge face mismatch between old and new rails.

The traditional process for precurving rail was the use of a "gag press," holding the rail at two points and using a hydraulic ram to place a tiny kink in the rail at an intermediate point. The process was repeated at intervals through the required length of the rail to produce a reasonable approximation of the desired circular curve radius. This process has largely been replaced by roller bending equipment that still uses three points, but produces an absolutely uniform curve rather than a series of kinks. Regardless of the equipment used, it is typically not possible to get a true curve in the last 18 inches [about 0.5 meter] of the rail. On extremely sharp radius curves, typically anything sharper than a 100-foot [30-meter] radius, it is therefore usually necessary to crop off these straight ends so that the joints are not kinked. For much the same reason, in bolted rail construction, it is often necessary to precurve the joint bars for extremely tight curves.

It is typically possible to spring long strings of CWR to fairly tight radii, and some track constructors will use that as a reason why they don't need to precurve the rail. However, springing the rail leaves it in a state of high internal stress. Obviously, the sharper the curve, the higher the stress, and that stress can make track maintenance more difficult. For example, a sprung curve in ballasted track is more likely to develop a sun kink at a single weak spot in the ballast section. The same curve using precurved rail will maintain line better. A broken rail that had been sprung into alignment may be nearly impossible to fix since the maintenance staff could have great difficulty getting the rail ends to line up squarely so that they could be rejoined by either a bolted joint or a field weld.

Some contractors have brought roller rail bending equipment onto jobsites and precurved previously welded strings of CWR for immediate installation in track. This method eliminates both the need to crop straight rail ends and the use of thermite welds and can thereby save both time and costs, provided the jobsite provides sufficient room to do the work.

5.2.2.6 Procurement of Tee Rail

Procurement of rail should be done in accordance with AREMA Chapter 4, Part 2, Section 2.1, supplemented by any specific requirements of the transit project. A major consideration for rail procurement is the proposed methods for shipping, handling, and welding. These issues should

be thought through before finalizing the rail procurement methodology and specifications. See Chapter 13 for additional considerations about procurement of rail and other track materials.

5.2.3 Groove Rail

While tee rail can be and is commonly used in embedded track, groove rail is arguably the preferred rail section for such construction. As the name implies, groove rails have a preformed flangeway in their top surface. On one side of the flangeway is the head of the rail, where the rail vehicle wheel treads roll in the usual manner. On the opposite side of the flangeway is a thinner segment of rail steel, variously called the "tram," "lip," or "guard." The tram defines the inner edge of the flangeway and, in embedded track, conveniently excludes the adjoining pavement material from encroaching on the flangeway.

Such rails were popularly known as "girder rails" in North America, but have not been rolled in the United States since the 1980s. Overseas, they are known as "groove rails" in the English language. The equivalent term in German is *rillenschiene.* The French call these rails *rail à orniére* or *rail à gorge profonde.* This Handbook will use the English version of the European terminology unless specifically referring to one of the girder rail sections formerly rolled in the United States.

5.2.3.1 Advantages of Groove Rail for Embedded Track

Groove rail has two principal advantages for use in embedded track:

- The preformed flangeway eliminates the tedious process of forming a flangeway in the embedding pavement. Instead, the constructor need only screed and finish the pavement between the trams of the parallel groove rails. This translates directly into construction cost savings by eliminating an appreciable amount of labor.

- It is easier to achieve a high level of electrical isolation using groove rail than it is with tee rail, especially when using a rail boot for isolation. The flangeways in embedded track, particularly track that is longitudinally level or only on a slight gradient, tend to fill up with the dirt, grit, and debris that is endemic to any street environment. This detritus, especially when wet, can be electrically conductive. When a flangeway is formed adjacent to tee rail, the debris will bridge the top of the rail boot in the floor of the flangeway, resulting in trace amounts of stray traction power current. However, groove rail can be completely wrapped in the rail boot on both sides of the rail, and stray current leakage can be appreciably less.

Groove rail has a distinct advantage when it is deemed desirable to use paving stones or bricks in the track area so as to achieve some architectural goal or ambiance. Such pavers can be laid directly up against the booted rail without going to extra measures to keep them from encroaching on the flangeway. See Chapter 4, Figure 4.7.13, for an alternative detail using brick pavers with tee rail.

5.2.3.2 Available Groove Rail Sections

The European Committee for Standardization (in French, *Comité Européen de Normalisation,* hence the usual abbreviation "CEN") has developed European Norm specification EN 14811 that

lists and illustrates the groove rail profiles available and stipulates requirements for their manufacture. There are about a dozen groove rail sections still being manufactured, most of which have been adopted as standard sections under the European Norms. Several of these sections are older designs that were kept in the Norms only because some transit agencies were still using them and apparently disinclined to change to newer designs. Because of this, not all the groove rails shown in EN 14811 should be considered for North American use. In most cases, use of these groove rails is not suggested due to gauge corner radii that are dramatically inconsistent with 115 RE tee rail and flangeways that are too narrow for most wheel flanges in use in North America. Not all groove rail sections are available from all European rolling mills, and some mills offer proprietary post-rolling treatment processes. Readers should confer with North American sales representatives of these rolling mills for current information concerning available sections and associated technical data.

As of 2010, five CEN groove rail sections are in common use in North America. These are listed below with both the current CEN identifiers and the names by which they were formerly known:

- Section 51R1 (formerly Ri52N) shown in **Figure 5.2.2.**
- Section 53R1 (formerly Ri53N) shown in **Figure 5.2.3.**
- Section 59R2 (formerly Ri59N) shown in Figure 5.2.2.
- Section 60R2 (formerly Ri60N) shown in Figure 5.2.3.
- Section 62R1 (formerly NP4aS) shown in **Figure 5.2.4**.

In addition, one other groove rail is worthy of consideration since it is one of only a few sections that can easily accommodate AAR wheel profiles and gauges: Section 67R1 (formerly Ph37a) shown in Figure 5.2.4.

So as to simplify the images, the dimensional information provided in Figures 5.2.2 through 5.2.5 has been abbreviated so that only key dimensions are shown. Complete dimensional data are readily available from vendors or by consulting EN 14811.

The flangeway in 67R1 is dramatically wider than the flangeways of the other groove rail sections. Nevertheless, despite first appearances, the 67R1 flangeway is actually in compliance with the rules set out in the Americans with Disabilities Act Accessibility Guidelines (ADAAG) for LRT flangeways in pedestrian areas. The CEN 46G1 and 68R1 sections can also accommodate railroad wheel flanges and gauges, and the former has the advantage of being somewhat shorter than the 67R1 section. Nonetheless, neither the CEN 46G1 section nor the 68R1 section are as common as the 67R1, and sources of supply are more limited.

Groove rails that could reliably be used as a restraining rail are limited. The following three sections are suggested based on the thickness and height of the tram.

- Section 56R1 (formerly Ri1c), as shown in **Figure 5.2.5**, has a raised tram and more closely resembles the former ATEA and AREA girder guard rails than any other groove rail section. However, it has a very small (6-millimeter [0.24-inch]) gauge corner radius and an exceptionally narrow flangeway. Unless a correspondingly small wheel profile is used, the 56R1 section likely can be used in North American LRT tracks only by

extensively machining the groove to increase the gauge corner radius and widen the flangeway.

- Section 62R1 (formerly NP4aS), shown in Figure 5.2.4, has been used successfully by three legacy streetcar systems in North America, all of whom use small wheel flanges. However, the narrow flangeway and steep guard face are far less than optimal compared to the former North American girder guard rail sections.

- Section 76C1, shown in Figure 5.2.5, is "construction rail" section most commonly used for fabrication of special trackwork components such as flange-bearing frogs. At some expense, it would be possible to have a customized flangeway of whatever shape milled into the head of this section.

Other groove rail sections listed in CEN Standard EN 14811 include 46G1, 52R1, 55G1, 55G2, 57R1, 59R1, 60R1, 60R3, 62R1, 63R1, 68R1, and 68G1. These are not recommended for North American use due to small gauge corner radii and insufficient flangeway width or depth. The 55G1 and 55G2 sections have a base width equal to that of 115 RE tee rail, but the base thickness and slope are radically different than 115 RE. Also, the relationship between each rail's gauge line and web vertical axis is appreciably different, so the common base width is of little advantage when considering rail fastening systems.

EN 14811 includes several other sections of rail that are classified as "construction rail profiles." These sections are primarily used as base materials for fabrication of special trackwork components. Several other groove rail sections are rolled that were not adopted as CEN standards. For example, there is a GP41 section that is very similar to the CEN-adopted GP35 section but has a 41-mm (1.61-inch) groove as opposed to the 35-mm (1.38-inch) groove of GP35.

The selection of groove rail currently available is limited to the European standards listed above. To use these narrow flange girder rails, the wheel gauge and track gauge must be compatible with a reduced gauge clearance between wheel and rail to allow for wheel passage. The wheel flange profile may need to be customized. For additional information on wheel profiles and groove rail, refer to Chapters 2 and 4.

5.2.3.3 Groove Rail Head Profile Compatibility with Tee Rails

Wheel compatibility based on head radii and wheel contact zone is possible if the wheel profile is designed to suit both tee rail and girder rail sections. The vehicle/wheel designer and the track designer must consider the impacts of wheel/rail performance resulting from standardized rail sections. For additional information on wheel/rail conformance refer to Chapter 2.

When the former Ri59 and Ri60 rail sections were redesigned to incorporate a 13-mm [about ½-inch] gauge corner radius, the crown of the head was also reprofiled. Together, these changes achieved a close compatibility with the former S49 (now 49E1) Vignole rail section, which is the predominant tee rail used in open track areas of European tramways and Stadtbahn systems. Coincidentally, this change in the groove rail design also made them, when fastened to a flat base, a reasonably good match with the original design 115 RE tee rail when it is installed at a standard cant of 1:40.

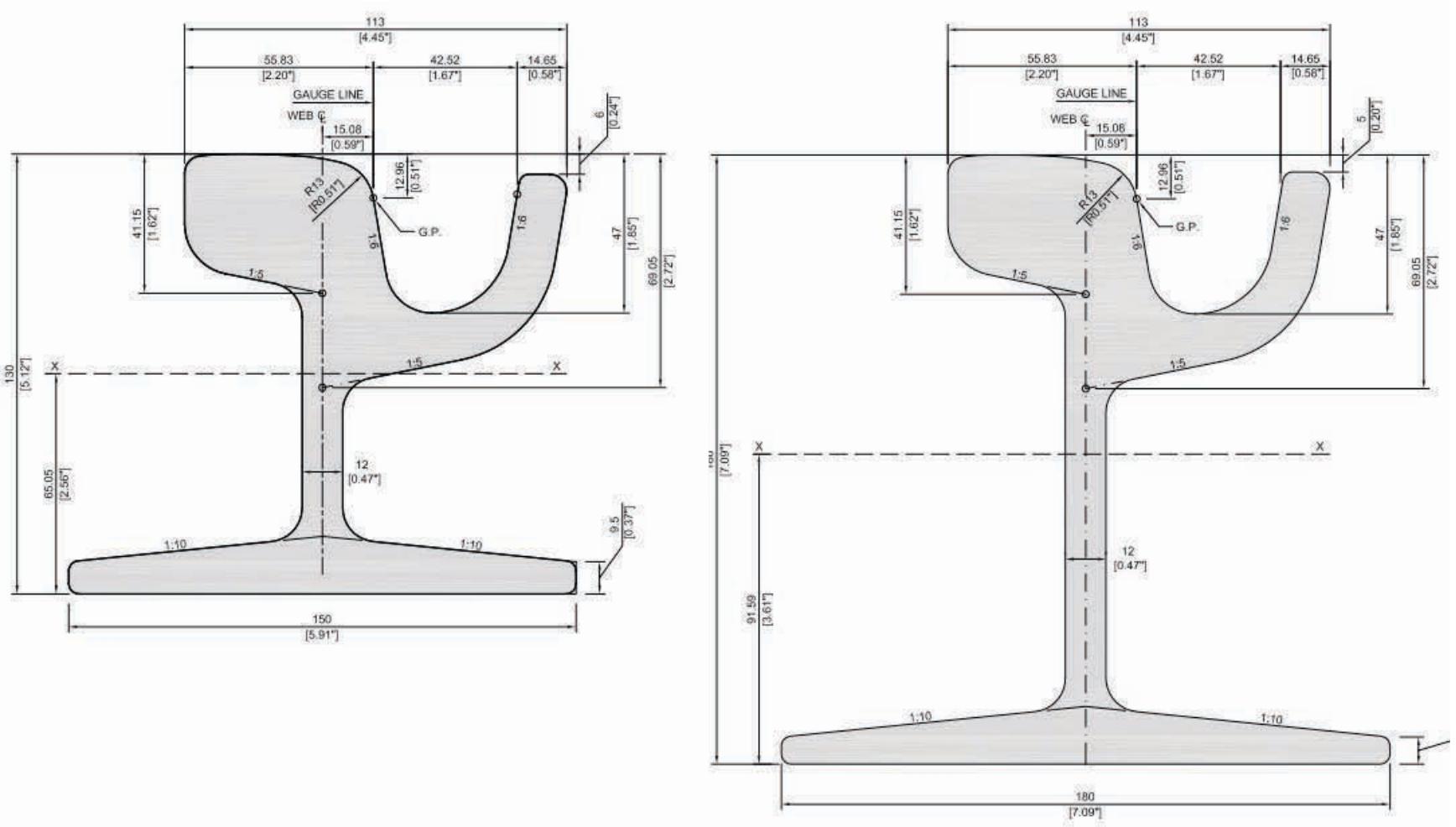


Figure 5.2.2 CEN 51R1 and 59R2 groove rail sections

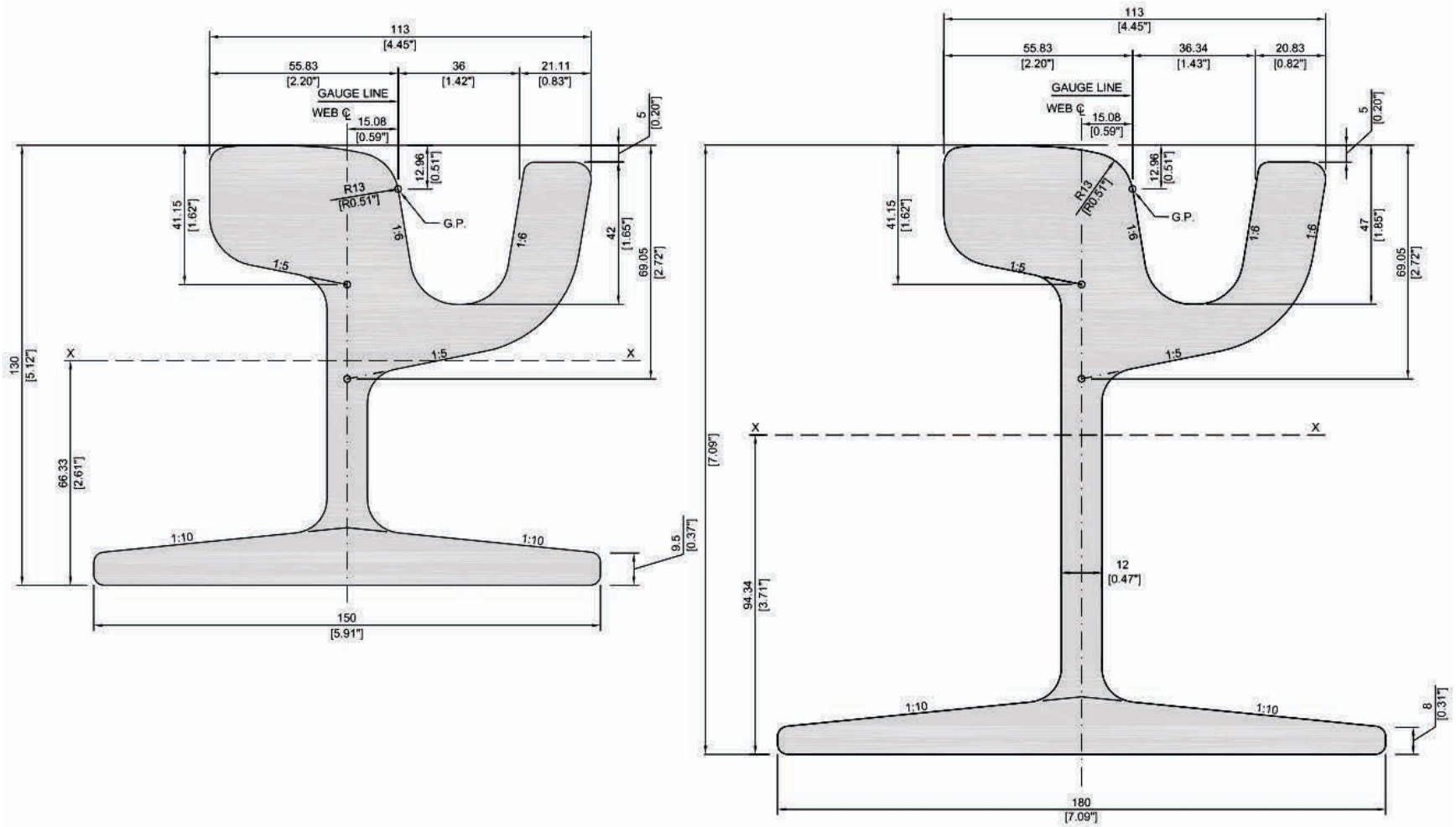


Figure 5.2.3 CEN 53R1 and 60R2 groove rail sections

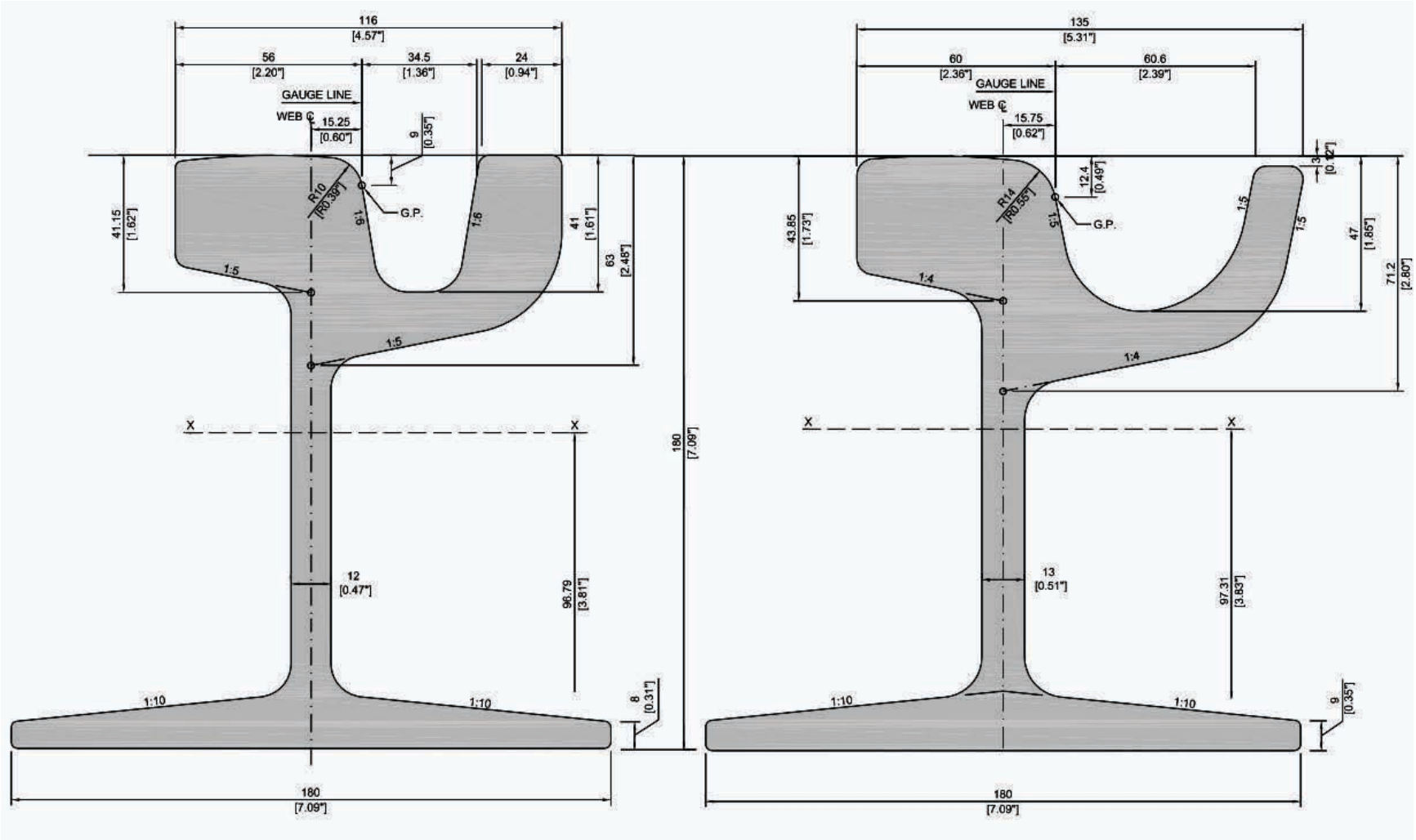


Figure 5.2.4 CEN 62R1 and 67R1 groove rail sections

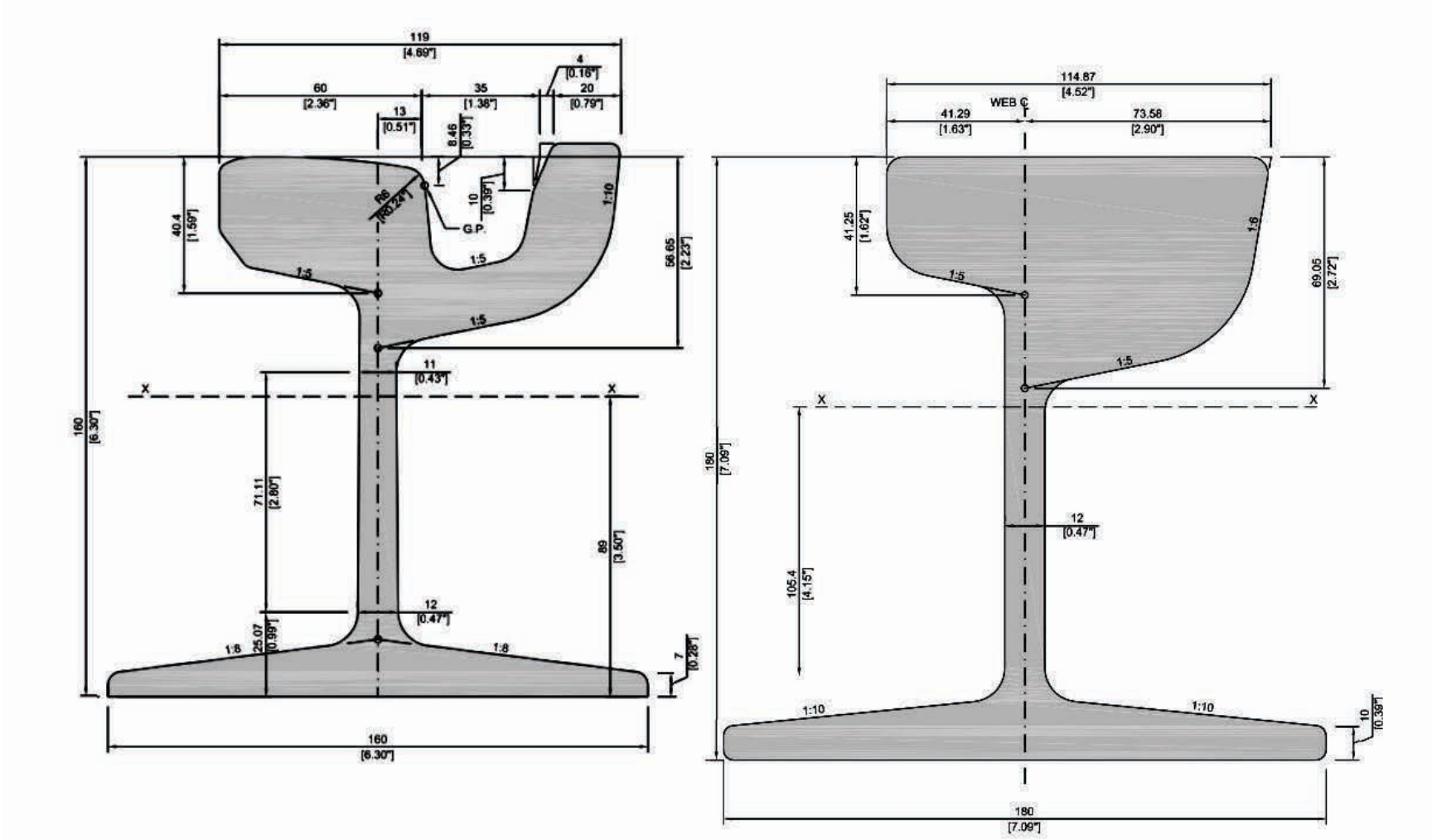


Figure 5.2.5 CEN 56R1 groove rail and 76C1 construction rail sections

Until 2009, the 115 RE rail section, which was first rolled shortly after World War II, included a 10-inch (254-millimeter) crown head radius. That radius is a reasonably good match to popular European groove rail sections that have a 300-mm [11.81-inch] crown radius. The 300-mm radius matches the 49E1 Vignole rail section often used on European light rail systems. However, in 2009, AREMA modified the design of the 115 RE rail to a crown radius of 8 inches, [203 mm] so as to match other modern North American tee rails, such as a the 141 RE section. The smaller radius is designed to better handle high loadings from average worn freight car wheels. With this change to the head radius of the 115 RE section, compatibility with European groove rails has been diminished.

Nevertheless, a smaller crown radius reduces the contact band along the rail to a well-defined ½- to 5/8-inch [12- to 15-millimeter] width. Light rail transit operations that have relatively little or no groove rail in track will likely benefit from this reduced contact band, as it can help control truck hunting in tangent track. Several transit agencies have incorporated more radical improvements, such as asymmetrical rail grinding (See Chapter 4, Article 4.2.5.3) for outside and inside rail in track curves, with documented operational improvements in wheel/rail performance. For additional information on rail grinding refer to Chapter 9, Article 9.2.1.3.3, and Chapter 14, Article 14.6.2.2.

When a groove rail track system is designed to match transit wheel profiles and gauges, off-the-shelf track construction/maintenance-of-way equipment (constructed to AREMA and AAR standards) will usually not fit the track. Damage to new track has been experienced under these circumstances.

5.2.3.4 Groove Rail Strength and Chemistry

The customary European steel manufacturing practice is to roll standard groove rail sections in accordance with the current CEN EN14811. The standard groove rails are produced in many grades of rail steel as shown in **Table 5.2.1**. Each designation classifies the groove rail per chemical composition by percent of mass, tensile strength, elongation in percent, and hardness. Hardness is designated as "HBW," which is equivalent to the Brinell Hardness Number (BHN) customarily used in North America.

To meet North American requirements for surface hardness a groove rail section must have a minimum tensile strength of 1,131 MPa (N/mm^2), which equates to approximately 340 BHN according to **Table 5.2.2**. So as to match AREMA high-strength tee rail, an optimal CEN groove rail steel designation from Table 5.2.1 would be R340GHT, a rolled rail section with a minimum tensile strength of 1175 MPa or 1175 N/mm^2, having a hardness of 340 to 390 BHN. Groove rail of this hardness can be obtained only through a combination of rail chemistry and post-rolling hardening procedures and may not be available from all manufacturers.

Since groove rail is a manufactured product, its metallurgy is subject to changes and improvements. Manufacturers also have proprietary processes that may be beneficial for specific LRT projects. Users of this Handbook are encouraged to confer with sales representatives of the various competing steel companies for current information.

Table 5.2.1 Chemical composition of CEN groove rail steel

CEN Steel Designation	Percent by Mass of Liquid Steel					H, max in PPM	Tensile Strength, MPa, min	Running Surface Hardness, HBW
	C	Si	Mn	P (max)	S (max)			
R200	0.40 to 0.60	0.15 to 0.58	0.70 to 1.20	0.035	0.035	3.0	680	200to 240
R220G1	0.50 to 0.65	0.15 to 0.58	1.00 to 1.25	0.025	0.025	3.0	780	220 to 260
R260	0.62 to 0.80	0.15 to 0.58	0.70 to 1.20	0.025	0.025	2.5	880	260 to 300
R260GHT	0.40 to 0.60	0.15 to 0.58	0.70 to 1.20	0.035	0.035	2.5	880	260 to 300
R290GHT	0.50 to 0.65	0.15 to 0.58	1.00 to 1.25	0.025	0.025	2.5	960	290 to 330
R340GHT	0.62 to 0.80	0.15 to 0.58	0.70 to 1.20	0.025	0.025	2.5	1175	340 to 390

Table 5.2.2 Brinell and Rockwell hardness related to tensile strength

Brinell Indentation Diameter (mm)	**Brinell Hardness Number**		**Rockwell Hardness Number**		**Rockwell Superficial Hardness Number, Superficial Diamond Penetrator**			Tensile Strength (Mpa)= (N/mm^2)*
	Standard Ball	Tungsten Carbide Ball	B Scale	C Scale	15-N Scale	30-N Scale	45-N Scale	
2.50		601		57.3	89.0	75.1	63.5	2262
2.60		555		54.7	87.8	72.7	60.6	2055
2.70		514		52.1	86.5	70.3	47.6	1890
2.80		477		49.5	85.3	68.2	54.5	1738
2.90		444		47.1	84.0	65.8	51.5	1586
3.00	416	415		44.5	82.8	63.5	48.4	1462
3.10	388	388		41.8	81.4	61.1	45.3	1331
3.20	363	363		39.1	80.0	58.7	42.0	1220
3.30	341	341		36.6	78.6	56.4	39.1	1131
3.40	321	321		34.3	77.3	54.3	36.4	1055
3.50	302	302		32.1	76.1	52.2	33.8	1007
3.60	285	285		29.9	75.0	50.3	31.2	952
3.70	269	269		27.6	73.7	48.3	28.5	897
3.80	255	255		25.4	72.5	46.2	26.0	855
3.90	241	241	100.0	22.8	70.9	43.9	22.8	800
4.00	229	229	98.2	20.5	69.7	41.9	20.1	766
4.10	217	217	96.4					710
4.20	207	207	94.6					682

* 1 MegaPascal = 1 Newton/$millimeter^2$

As a guideline, groove rail should have a running surface hardness not less than 340 BHN. Higher hardness is difficult to obtain in groove rails since the asymmetric shape of the rail makes it difficult to head harden by conventional heat treatment methods. However, at least one

European rail manufacturer has developed a proprietary head hardening process for groove rail. Their rail is categorized as Head Special Hardened (HSH) and has a test hardness of about 365 BHN in the rail head and tram (lip) of the groove section. Their product was utilized on one LRT system in North America with 100% of the main tracks built using HSH groove rail. After several years of operation, the rail shows exceptional resistance to corrugation and very little wear.

5.2.3.5 Precurving of Groove Rail

Like tee rail, groove rail must be precurved if the curve radius is sharp enough to make springing the rail impractical.

The radius thresholds for precurving groove rail are generally higher than for tee rail. This is because the asymmetrical shape of the groove rail causes it to curl when sprung horizontally so that the base no longer lies flat. For this reason, it is often necessary to camber groove rail vertically prior to bending it horizontally, particularly when using a gag press rail bending method. The amount of camber will vary by both the rail section and the horizontal radius desired. The direction and amount of camber will vary depending on the rail section and whether the rail is on the inside or outside of the curve. Typically, inner rails will require a negative camber (a "smile") while outer rails will require a positive camber (a "frown") prior to being curved horizontally. Cambering is sometimes not necessary when precurving groove rails using a roller bender that tightly clamps the rail to constrain vertical movement.

General guidelines for precurving groove rails are the following:

- Horizontal Curves—precurve groove rail for curve radii below 450 feet [137 meters].
- Vertical Curves—precurve girder rail for vertical curve radii below 1000 feet [300 meters].

5.2.3.6 Procurement of Groove Rail

Procurement of groove rail requires specific contract language stating the requirements as to rail section, strength, special treatments, and potential precurving requirements in specific lengths of rail. The incorporation by reference of the most recent version of CEN European standard EN14811 is acceptable as long as additional special provisions are included.

As a guideline, the special provisions section for procurement of groove rail should include the following:

- The ultimate tensile strength of the rail to be supplied, in particular, the minimum Brinell Hardness Number at the two wearing surfaces—the groove rail head and tram.
- The compatibility of welding and providing guidelines as to welding—both electric flash butt and thermite welding.
- Precurving requirements, including specific length of rails.

Since, as of 2010, groove rail is only rolled in Europe, delivery to a project in North America will involve several stages of handling and transport by multiple parties, including transatlantic shipment. So responsibilities for any damage are clear, it is recommended that specifiers avoid specific handling methods and instead focus on the condition that the rail must be in upon

delivery to the project. It should be very clear that acceptance and payment for the rail is conditional on meeting the stated requirements.

Federally funded rail transit projects in the United States are subject to the requirements of 49 CFR 661, commonly known as "Buy America." Briefly stated, "Buy America" provisions stipulate that products purchased for construction of a federally funded transit project must have been made in the United States unless either (1) the item is not made in the United States or (2) the item is not made in the United States in sufficient quantities to satisfy the needs of the project. Project owners wishing to procure a foreign-made product under either of those exemptions must first obtain a waiver from the Federal Transit Administration. Up until 2010, such waivers were routinely granted for procurement of groove rail since no comparable product was made here. However, during 2010, the FTA began to interpret "Buy America" very strictly and waivers for groove rail were no being longer granted. As of this writing, it is unclear whether this restrictive policy will continue. Project owners that are considering the use of groove rail should very closely consider whether their funding mechanisms will permit them to do so. In that regard, it should be noted that similar restrictions may be attached to state or local funding.

5.2.3.7 Block Rail

Late in 2010, as a direct result of the "Buy America" issue mentioned in the paragraph above and as this second edition of the Handbook was undergoing final editing, it was announced that a variation of groove rail would be manufactured in the United States. Called "block rail," this product effectively eliminates the customary web of the rail and places the head and tram directly on the base of the rail. **Figure 5.2.6** illustrates a typical block rail section.

Photo courtesy of HDR Engineering

Figure 5.2.6 LK1 block rail section

There are at least three distinct sections of block rail rolled in Europe, where it has been used for decades for two purposes:

- Fabrication of temporary tracks, particularly temporary crossover tracks that sit on top of existing embedded trackwork.
- Construction of modular embedded tracks using precast concrete panels. This was fairly common in the former Soviet Bloc countries up through the 1990s, but is reported to have fallen out of favor with some (but not all) transit agencies there due to maintenance issues with existing installations.

As of this writing, an initial rolling of block rail has occurred at a steel mill in Pennsylvania and some block rail has been installed on a streetcar project in Portland, Oregon. Full details of that installation are, as of this writing, not yet public; however, preliminary information presented at an APTA conference in June 2011 suggests that details used in Eastern Europe are not proposed.

Some industry observers have noted that block rail has a very low section modulus and hence has very limited beam strength. Because of that, block rail likely should be continuously supported so that the sinusoidal wave action response that all rails have to a rolling load does not result in a permanent vertical warping of the rail. Photographs of block rail installations in Eastern Europe suggest that this problem may be common. The Portland installation will therefore generate great interest as it is designed, built, and brought into revenue operation.

Handbook users who believe they might have an application for block rail are strongly encouraged to consult recent trade publications and obtain current information from manufacturers before suggesting the concept to their project management.

5.2.4 Rail Wear

Rail continually suffers from abrasive wear due to the steel wheel running on and against it. Surface head wear is due to the constant running of the wheels and is further compounded by the additional forces generated by braking and traction during deceleration and acceleration, respectively. In curved track, there is added surface wear, where wheel slippage and load transfers occur due to the changing direction of the vehicle truck. Gauge corner and eventually gauge face rail wear occur due to the steering function of the rail. Steering contact is at the outer rail of a curve, which guides the outside wheel of the lead axle. The action commences when the vehicle wheels negotiate the outside rail of the track curve to the point where the wheel flange makes contact with the gauge corner due to the designed freeplay between the wheels and the rail head. Because of the freeplay, the wheel contact is virtually never normal to the direction of the rail, but at a slight angle that is referred to and measured as the "angle of attack."[4]

This attack on the outer rail is not caused by the vehicle's centrifugal force, but by the constant change in the track alignment that results in changing the vehicle's direction. The outer rail constantly steers the outer leading wheel inwards towards the curve center. This function requires the truck to rotate ("skew" or "yaw"), and, in a conventional truck, that rotation is resisted by friction in the connections between the truck and the car body. For additional information on truck skew, refer to Chapter 4 and the discussions concerning Nytram plots at Article 4.2.4.

The wheel acts as a cutting tool, or grinding stone, that actually machines the steel at the gauge corner and eventually the face of the running rail. This is caused by several factors, such as the severity of the wheel's angle of attack to the rail, the friction between wheel and rail, and the

stiffness against rotation of the vehicle truck. The latter increases the force against the rail, increasing friction and wear, and concurrently reduces the speed of the vehicle.

Another rail wear phenomenon is the formation of metal flow. The wheel/rail interaction causes the rail and steel surfaces to deform at the point of contact due to the concentrated load. This contact pressure is extreme to the point where the stress is greater than the yield point of the rail steel, which causes plastic deformation of the surrounding surface steel. This action leads to metal flow accumulation on the surface edges of the rail head. Metal flow may collect at the gauge corner of rail in tangent track, where the wheel is seldom in contact with the rail gauge corner or face. Metal flow collection also occurs on the field side of the inside rails in curves, where the rail head metal flow migrates toward the field side and accumulates as a pronounced lip. Slivers of metal flow have been known to eventually wear through completely, forming a long slender metal strip that drops off the rail, generally at the base of the gauge side rail head due to full depth rail gauge face wear. This condition is far less prevalent in transit track than it is on freight railroads, but, as rail ages, the possibility of metal flow grows and this is with the lightest of vehicles.

Corrugation of rail is another rail wear phenomenon that severely impacts ride quality and noise generation. Corrugation is discussed in detail in Chapter 9, Article 9.2.1.1.3.

5.2.5 Wear-Resistant Rail

Transit systems have historically suffered from worn rails and the need for rail replacement due to accumulative wear limits of the rail head and/or gauge face. To combat the wheel machining of the rail gauge face and loss of metal, an abrasion-resistant steel is required. Improvements in the chemical composition and treating process of rail steel have led to the development of wear-resistant grades of steel. Research has shown that a fine grade of tempered pearlitic steel with sufficient hardness will be resistant to both wear and abrasion (or machining) of the steel and the formation of corrugations.[5]

The hardness of rail steel is proportional to its toughness or its ultimate tensile strength (UTS). UTS is used to measure the quality of the steel.

Up until about the mid-1990s, European groove rails, like North American girder rails, were rolled from very soft steel. Typical surface hardness was in the range of 220 to 240 BHN. Soft metallurgy was used because groove rails need to make more passes through the rolling equipment than tee rails of similar weight. Since the string loses heat during each pass, the chemistry needed to be soft so that the steel would stay sufficiently ductile for the last pass through the rolls, when the tram is bent up to form the flangeway. However, because the metallurgy was soft, such rails wore rapidly, particularly under the heavier loadings of modern light rail vehicles. In response to this rapid wearing of rails, European steel companies developed the harder grades of steel that are now available for groove rail.

Prior to the development of harder grades of steel, proprietary processes for head hardening groove rails were sometimes used to provide a more wear-resistant surface to the rail. For example, a special surface weldment known as Riflex[6], which also featured anti-squeal characteristics, was popular circa 2000. However, Riflex and similar processes were expensive.

Such post-manufacturing treatments are now seldom used as more wear-resistant grades of groove rail are now readily available directly from the rail mills.

5.3 RESTRAINING RAIL DESIGNS FOR GUARDED TRACK

Guarded track in light rail transit design, as described in Chapter 4, Article 4.3, can reduce outer rail gauge face wear on sharp curves by restricting movement of the wheels toward the outer rail. The guard (or restraining) rail is positioned close to the inside rail of the curve and contacts the back of the inside wheel flange. In an optimal condition, steering action can be realized at both wheels of the axle.

The designs of restraining rails can differ dramatically between projects, and over the years various designs have been used. Traditionally, curve guarding on North American street railway systems was usually achieved using a girder guard rail section somewhat similar to the 56R1 groove rail section illustrated in Figure 5.2.5. Ballasted and direct fixation track requiring guarding typically used a separate restraining rail mounted adjacent to the running rail. However, then as now, exceptions can be found, depending on the requirements and circumstances of a particular light rail system.

Sharp curves with restraining rail are very complicated to design, fabricate, and construct in the field. Prefabrication of restraining rail curves, including full assembly on a shop floor for initial acceptance inspection, can improve quality and reduce field installation time by detecting and correcting fabrication errors.

The following articles discuss the various designs and hardware for providing guarded track.

5.3.1 Groove Guard Rail for Embedded Track

Groove rail can be employed for two purposes:

- To provide a uniform flangeway in embedded track without needing to form one in the pavement.
- To provide a restraining rail that is integral with the running rail, thereby simplifying the design by eliminating one rail and the associated mounting and connection hardware.

These two purposes are not necessarily linked. It can be desirable to use a groove rail for curve guarding purposes even if tee rail is used in the same track segment when guarding is not required.

5.3.1.1 North American Girder Guard Rail—Background

While dozens of different girder guard rail sections were once rolled in North America, the most common sections after about 1930 were the 140ER7B and 152ER9B sections (designed for transit use) and the 149RE7A section (designed for use with railroad wheel flanges). These sections were developed specifically for embedded street track and North American wheel profiles and provided a robust tram on the side of the flangeway to act as a working guard face. Unfortunately, these sections are no longer rolled. The last North American rolling mill to produce them got out of the business in the mid-1980s when the rolls used to produce girder rail were too

worn to meet quality requirements. For various reasons, the product line was not sufficiently profitable to justify the investment in new rolls, so production ceased.

Once the North American girder rail sections were no longer available, transit agencies turned to European groove rail sections. Up through about 2000, the most popular European sections were Ri59 (now known as 59R1 and 59R2), Ri60 (now 60R1 and 60R2), and a section known as GGR-118, which is no longer available. These sections were all designed solely for transit vehicle wheels with relatively small flanges. Other groove rail sections rolled in Europe that can be considered for transit use in North America are listed in Article 5.2.3.2 of this chapter. With the exception of the CEN 67R1 section (formerly Ph37), the European groove rails shown are not compatible with freight operations.

5.3.1.2 Restraining Rail Issues with CEN Groove Rails

With the exception of the CEN 67R1 section (and two other similar non-CEN rails), the European groove rail sections are adaptable to North American use only if a transit wheel gauge is selected for the wheel set. The AAR wheel gauge of 55.6875 inches [1414 millimeters] is not compatible with the other groove rail sections. However, CEN 67R1 is not configured as a restraining rail. The flangeway, 58.66 millimeters [2.309 inches], is too wide and the tram, 16.31 millimeters [0.642 inch] thick and positioned 3.00 millimeters [0.12 inches] below top of rail, is both too thin and too low. The other few available groove rail sections with wider flangeways are similarly configured.

The dilemma confronting the North American light rail track designers whose projects need to comply with AAR railroad gauge standards is the lack of a suitable groove rail section that has the increased flangeway width required to accommodate railroad wheel sets and has a curve guard for embedded sharp radius curves. Even if the freight railroad does not operate over a particular track segment, the standards used elsewhere on the system may set requirements that must be met in the embedded track area.

As a solution, consideration could be given to machining a flangeway of an appropriate shape into the head of one of the several “construction rail” sections available from European manufacturers. Primarily designed for fabricating special trackwork (such as flange-bearing frogs), these sections could be machined to just about any head profile desired. This method has been employed on some U.S. LRT properties, but it is expensive.

As noted in Chapter 4, Article 4.3, the use of restraining rail is technically optional. Many LRT operations, both abroad and in the United States, successfully operate without restraining rail, albeit with increased rail gauge face wear. As a guideline, for systems using a transit wheel profile with a transit wheel gauge of 56 inches (1422.4 millimeters), most CEN groove rail sections rolled and treated to the hardest grade of steel possible will generally provide satisfactory service in tangent track and flat radius curves. Service life on sharp curves can be expected to be appreciably shorter.

5.3.1.3 The Possibility of a New North American Groove Rail

There has been appreciable discussion in the industry as to whether a new design of groove rail could be made that more closely matches North American needs. For that to occur, there are two principal issues to be addressed and resolved:

- What should the new section look like?
- Where can it be rolled?

It is unlikely that any previously rolled section would be totally satisfactory, and detailed investigation would be necessary to make certain that all reasonable issues are addressed. The short-lived GGR-118 rail, which was intended for North American use, was deliberately designed with a 139.7 mm [5 ½ inch] wide base for nominal compatibility with 115 RE rail, but the thickness and top slope of its base were totally different. Accordingly, it was not possible to use the same rail fastening hardware. The tram was also too thin to be an effective restraining rail over the long term. Hence, resurrection of GGR-118 is not a likely solution. A new groove rail section is likely required, one with both a wide flangeway and a guard/tram that is thick enough to take decades of wear and is raised to at least the rail running surface.

Such a section might closely resemble the former 149RE-7A girder guard rail section with revisions to the shape of the head (for compatibility with wheels that spend most of their time running on canted 115 RE) and the height (for compatibility with 115 RE). Consideration might also be given to configuring the base so as to be compatible with rail fastenings used with 115 RE. If an organization such as APTA and/or AREMA Committee 12 were to undertake this project, it would probably be possible to work toward one or two designs that would meet the needs of the majority of North American LRT systems.

The “where” question is more problematic. North American steelmakers withdrew from the girder rail business because it wasn’t sufficiently lucrative to justify the investment. The current resurgence of LRT and streetcar projects is unlikely to change their perspective. Rail mills in the United States understandably prefer to continue catering to the needs and requests of their biggest customers—the freight railroads of North America. The rolling of block rail at a U.S. mill came about in part because the steel company involved politely declined to roll groove rail. They agreed to instead roll the block rail because it was both a relatively simple section to roll and, more importantly, they could roll it on existing machinery at a location other than their rail mill. Such experience suggests that pursuing the rolling of a groove rail in the United States is likely to be an exercise in futility.

The TCRP Project D-14 research team is under the impression that European rolling mills can be relatively cooperative concerning rolling of special sections for customers. An example is a non-CEN groove rail section, Ri5, which Voest Alpine rolled specifically for Yarra Trams in Melbourne, Australia. If consensus can be reached on an appropriate girder guard rail section for North American use, it might be possible to interest one or more European mills to provide quotations. However, the more significant problem is the “Buy America” issue discussed in Article 5.2.3.6.

5.3.1.4 Alternatives to Groove Rail for Guarded Embedded Track

Where curve guarding is desired in embedded track and groove rail is not used, alternate designs are available. The two most common details include

- Conventional vertically mounted tee rail restraining rail, similar to the details described in Article 5.3.2.1 of this chapter.

- Strap guard rail, bolted to 115 RE tee running rail, as described in Article 5.3.2.3 of this chapter.

5.3.2 Restraining Rail Options for Use with Tee Rail Construction

When groove rail is not used, but a restraining rail is desired, tracks with sharp curves have been equipped with various designs to provide the required restraint. Guarding is typically provided by mounting a separate "restraining rail" parallel and concentric to the inside running rail, with the horizontal distance between the two rail heads set at the required flangeway width dimension. See Chapter 4, Article 4.2.4.2, for guidance on determination of the appropriate restraining rail flangeway width using the Nytram Plot procedure.

The restraining rail can be fabricated from one of several steel shapes and may or may not be physically attached to the running rail. In versions that are physically bolted to the running rail, the restraining rail/running rail assembly must be designed as a unit so that curvature is consistent and bolt holes in both rails are aligned.

5.3.2.1 Vertically Mounted Restraining Rails

The most common type of restraining rail is a vertically mounted tee rail as shown in **Figure 5.3.1**. The restraining rail is fabricated by planing away a portion of the rail base of a standard tee rail, which is then bolted to the running rail at intervals of 24 to 36 inches [roughly 600 to 900 millimeters]. Cast or machined steel spacer blocks are placed between the running rail and the restraining rail to provide the desired flangeway. Some designs fabricate the spacer blocks in two pieces and insert shims between them to adjust the flangeway width so that the flangeway width can be restored to the design dimension as the guard rail face wears. Although this design feature appears sound, experience has shown that few transit systems actually ever take advantage of this maintenance feature.

The restraining rail and the running rail webs must be match drilled to insert connecting bolts. The bolt hole spacing must be detailed on the shop drawings because the restraining rail is on a slightly larger horizontal radius than the inside running rail to which it is attached. In addition, the bolt hole spacing will be different on each rail. While this differential is minor between any pair of bolt holes, it will become significant when accumulated over the full length of a rail.

For curves using 115 RE rail with radii less than 300 feet [91 meters], the combined running and restraining rails are typically precurved, fabricated, and assembled together on a shop floor. Each piece is numbered and match marked similar to special trackwork assemblies such as turnouts. For ease of shipment, these precurved segments are usually 36 feet [11 meters] long or shorter. It is suggested that the precurved running rail not be shop drilled to match the restraining rail. Instead, drilling can take place after the running rail is thermite welded in track and placed in final position, matching the as-built locations of the predrilled holes in the restraining rail. By working from one hole to the next, drilling and bolting the rails together in a continuous process, it is possible to keep the holes closely aligned. During field assembly, should any of the shop-drilled holes coincide with or be within 6 inches [150 mm] of a weld in the running rail, it is necessary to abandon the shop-drilled hole in the restraining rail and drill a new one at an appropriate longitudinal distance from the rail weld.

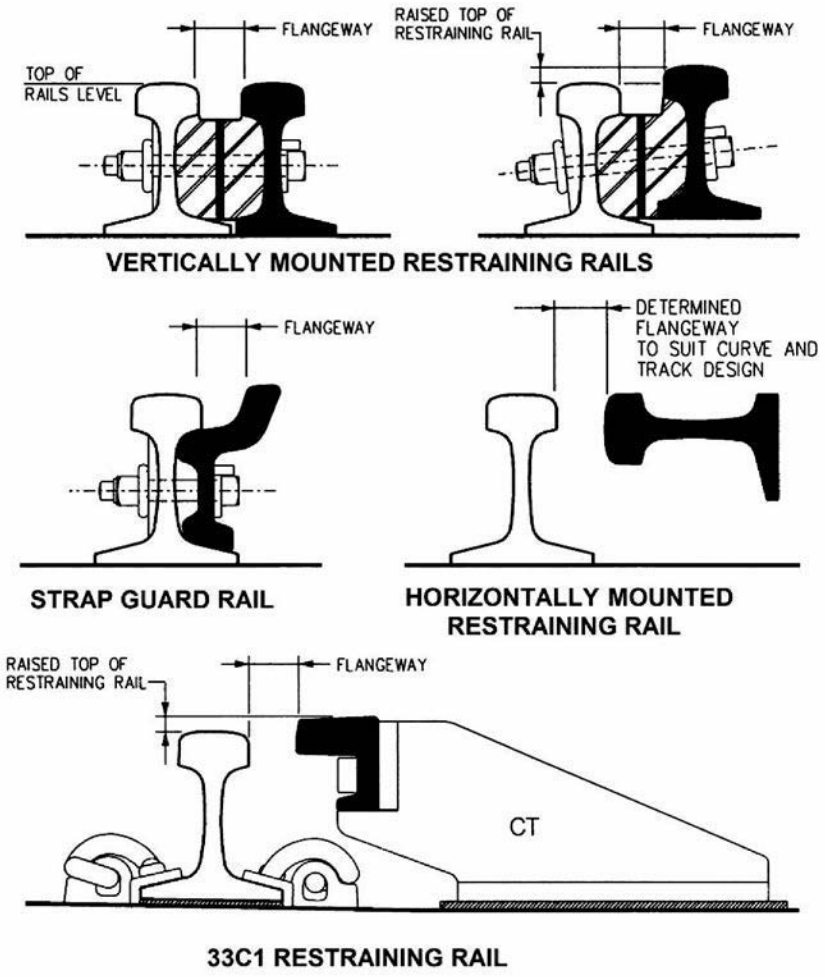


Figure 5.3.1 Typical restraining (guard) rail arrangements

For curves with radii greater than 300 feet [91 meters] and through curve spiral segments with instantaneous radii above that threshold, both the CWR running rail and the restraining rail can usually be field sprung to the desired curvature. In such cases, shop curving of both running and restraining rails is typically not essential.

The restraining rail is often the same rail section as the running rail. In cases where the restraining rail is elevated above the head of the running rail, the restraining rail is sometimes fabricated from the next larger rail section (e.g., 115 RE running rail would be paired with a 132 RE restraining rail). In other designs, the same rail section is used, but a riser shim is welded to the rail fastening plate beneath the restraining rail to elevate it. See Chapter 4, Article 4.2, for additional discussion concerning restraining rail configuration, including height, flangeway width, and guard face angle. See also *TCRP Research Results Digest 82*, which contains extensive discussion concerning restraining rail height and application.

If elastic rail fastenings are used, the spacing between the restraining rail bolts and the cross tie or rail fastener should be coordinated to ensure that the bolt assembly will not interfere with either insertion or removal of the elastic rail clip. Similarly, it should be possible to either tighten or remove the bolt assembly without removing the rail clip.

On timber cross ties, the combined running rail/restraining rail assembly will usually be installed on a common extended rail fastener or tie plate unlike those used under single running rails. Restraining rail installed on concrete cross ties will require a special restraining rail cross tie with a wider shoulder mounting. Some designers support the restraining rail only at every other cross tie or rail fastener location.

Restraining rails that are physically attached to the running rail can compromise broken running rail detection by providing an alternate path for signal circuit current around the break. Similarly, mounting plates shared by the running rail and restraining rail can create a circuit path around a rail break. Groove rail, as a single entity, does not have this problem since the guard face would be included with the break. A similar situation exists at running rail insulated joint locations. Both the running rail and the restraining rail must be designed with an insulated assembly. For vertically mounted tee restraining rail, there usually is a common double rail end post configuration that suits the flangeway width.

Vertically mounted restraining rails have been used in all the types of track structures. Nonetheless, when vertically mounted restraining rails are employed in embedded track, it is necessary to attempt to completely seal the flangeway to keep out moisture and accumulating debris. The insulating rubber rail boot in a two-rail configuration is available and can be included in the track design. A restraining rail assembly in embedded track will have multiple paths for water to seep into the boot. Even with sealants, it is critical to provide drainage to keep the rail enclosure relatively dry. Track drains with open rail boot areas at the transverse track drains can provide this drainage relief. See Chapter 4, Article 4.7.4.4, for additional discussion of embedded track drainage.

In embedded track, vertically mounted restraining rail has the problem of sealing the floor of the flangeway against water intrusion. In the absence of a seal, the flangeway (and the rail boot, if used) could fill up with storm water and debris, possibly leading to problems, particularly in cold winter climates. The depth of the open flangeway can also be a safety issue for pedestrians; see Chapter 4, Article 4.3.5.3, for additional discussion on this point.

5.3.2.2 Horizontally Mounted Restraining Rails

Transit systems have used horizontal designs where the restraining rail is mounted with the rail's Y axis oriented horizontally, as shown in Figure 5.3.1. This is a relatively old design that is currently seen only in older rail transit installations. Horizontally mounted restraining rail cannot be used in embedded track areas. The mounting hardware is bulky and expensive. Similar results can likely be achieved at a lower cost by using the 33C1 guard rail section discussed below. While some legacy rail transit systems used and/or still use horizontally mounted restraining rail in open trackforms, it is not suggested for new light rail installations.

5.3.2.3 Strap Guard Rail

An innovative restraining rail design uses a special rolled steel section known as "strap guard" with 115 RE rail (see **Figure 5.3.2**). The strap guard section is bolted directly to the web area of the running rail similar to a joint bar.

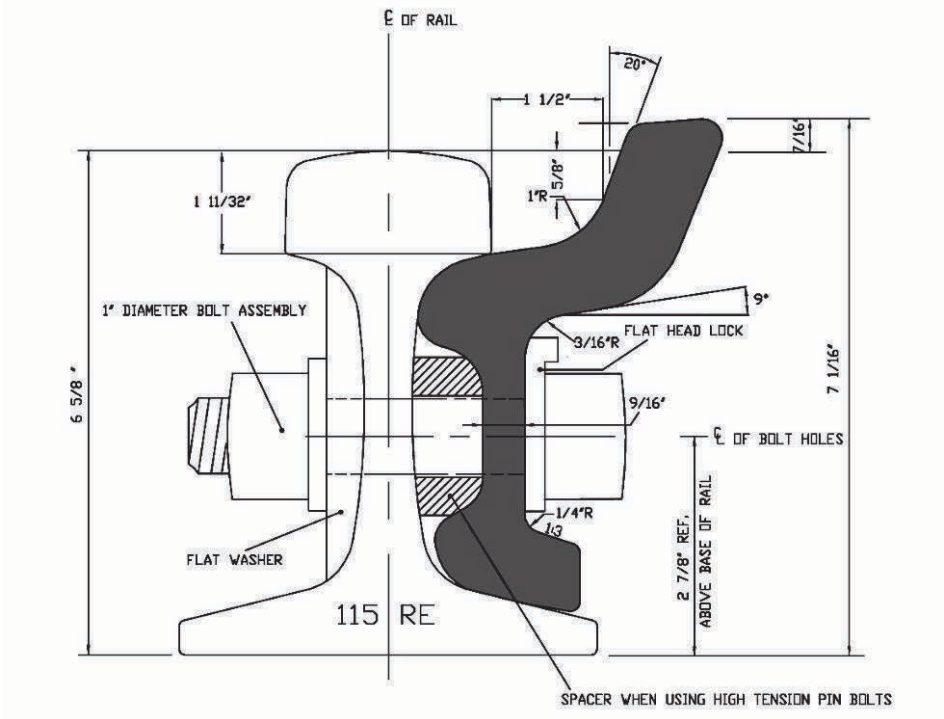


Figure 5.3.2 Strap guard rail

The strap guard section was developed for the Pittsburgh light rail transit system in the early 1980s based on similar sections that were rolled for use with ASCE tee rails in the early 20th century. Pittsburgh is the still the largest user of strap guard, using it in all trackforms, and considering nothing else for restraining rail. However, strap guard has seen limited application outside of Pittsburgh. There are short installations in Baltimore, Dallas, Galveston, Tampa, and Kenosha. Boston's MBTA tried a short section, but opted for a different design instead.

Advantages of strap guard include the following:

- It does not require special rail tie plates, rail fasteners or cross ties. The only requirement is a specially designed rail clip that can bear on the lower flange of the guard on the gauge side of the assembly. The field-side rail hold-down device can be the same as that used in ordinary single rail installations. This facilitates adding strap guard to an existing curve in open track that is experiencing unacceptable levels of gauge face rail wear.
- It is a universal guarding system that can be used in ballasted, direct fixation, and embedded track.
- In embedded track, when compared to a vertically mounted tee restraining rail assembly, strap guard has an advantage since the configuration provides a solid floor to the flangeway. By contrast, the opening in the flangeway area in the vertically mounted tee rail and restraining rail assembly requires a substantial amount of filler material to exclude moisture and address safety concerns.

- When considering rail breaks in areas of strap guarded rail, the strap guard rail actually provides some minimal security against both a wide gap and rail end mismatch since its shape effectively places a joint bar across the rail break. On the other hand, the resulting electrical continuity would mask a running rail break from the signal system by providing an alternate path for signal current around the rail break. Note that, in general, broken rail protection is not an issue in embedded track since the pavement structure will still keep the rail ends aligned.

Rail boot is commercially available for strap guard as well as some configurations of vertically mounted tee restraining rail assemblies.

Strap guard has several issues that must be carefully considered:

- As currently designed and rolled, strap guard mimics the flangeway dimensions of the former ATEA 140ER7B and 152ER9B girder guard rail sections and, as such, is really suitable only for small flanges such as the former ATEA designs. Baltimore shimmed out the strap guard to widen the flangeway to match an AAR wheel profile and gauge, but does not actually employ it as a restraining rail.
- Strap guard is presently rolled in maximum lengths of 30 feet [9.14 meters], a limitation of the rolling mill equipment where it is produced.
- The depth of the flangeway provided by strap guard is limited by the height of the head of 115 RE rail. For tall wheel flanges, running rail head wear could therefore limit the service life of the strap guard instead of wear on the working face of the guard. The issue could be relieved by using strap guard with 119 RE rail sections, but that would reduce the effective height of the guard. In addition, 119 RE has significantly dropped in popularity and is no longer an AREMA-approved design. Its long-term availability is uncertain.
- Strap guard, like many restraining rail designs, must be bolted to the running rail at frequent intervals. The bolt holes and bolt assemblies are potential maintenance issues, although Pittsburgh has not had any significant problems in this regard.
- The strap guard requires that the “collar” that results from thermite welding be ground flush with the fishing of the rail. Some thermite weld kit vendors strongly discourage this amount of rail surface weld finish, stating that the rail weld might actually be more prone to failure. No specific data are available to confirm or deny this assertion, and a substantial quantity of rail welds exist in LRT tracks that have been finish ground to that degree without breaks, tending to disprove this theory. Notably, many of the common concerns about thermite welds are based on experience under railroad loadings, and the substantially lighter loads of LRT make thermite weld failure appreciably less likely.
- While strap guard is not a proprietary design, it is currently made at only one rolling mill, the Arcelor Mittal facility in Steelton, Pennsylvania, coincidently the same mill and rolling machinery that has produced block rail. The rolls, which belong to the transit agency in Pittsburgh, will not fit on the equipment at any other rolling mill. When they eventually wear out, it is uncertain what the replacement cost and the source of funding will be. This situation is not unlike the former GGR-118 groove rail section, which was produced at

only one rolling mill in Europe. When restructuring of the European steel industry resulted in the closure of that specific rail mill, GGR-118 suddenly became unavailable.

Slight changes in the rolls and/or the rolling procedures for strap guard might permit the flangeway width and guard angle to be customized to match any given combination of wheel profile and curve radius. If the continuing availability issue noted above could be addressed in a comprehensive manner (perhaps through sponsorship of the design by a consortium organization such as APTA or AREMA), it might be possible to produce strap guard in different shapes so as to better suit the needs of LRT systems using large wheel flange profiles.

Some practitioners object to the elevation of the strap guard above the plane of the running rails, arguing that it could be both a tripping hazard to pedestrians and an impediment to snow plows. As discussed in Chapter 4, Article 4.3.5.3, elevated restraining rail of any type is generally discouraged in pedestrian areas, but that limitation should not affect restraining rail design in areas where there is no legitimate pedestrian traffic. Snow plow blades are routinely designed to deal with irregular pavement surfaces, including deviations appreciably larger than the height of the strap guard rail. Where track with strap guard is crossed by a designated pedestrian path, it is a simple matter to machine the top of the guard so as to be level with the running rail.

5.3.2.4 33C1 Restraining Rail

A different type of restraining rail design becoming popular in North American light rail transit design is the 33C1 section (see **Figure 5.3.3**) currently rolled in Europe. Prior to its adoption as part of the European Norms, the 33C1 section was referred to as either UIC-33 or U69 in the French and German standards, respectively. The 33C1 section in Europe was developed and has primarily been used as a guardrail for special trackwork frog locations. More recently, 33C1 has also been used for frog guardrails and continuous restraining rails on several North American light rail transit systems.

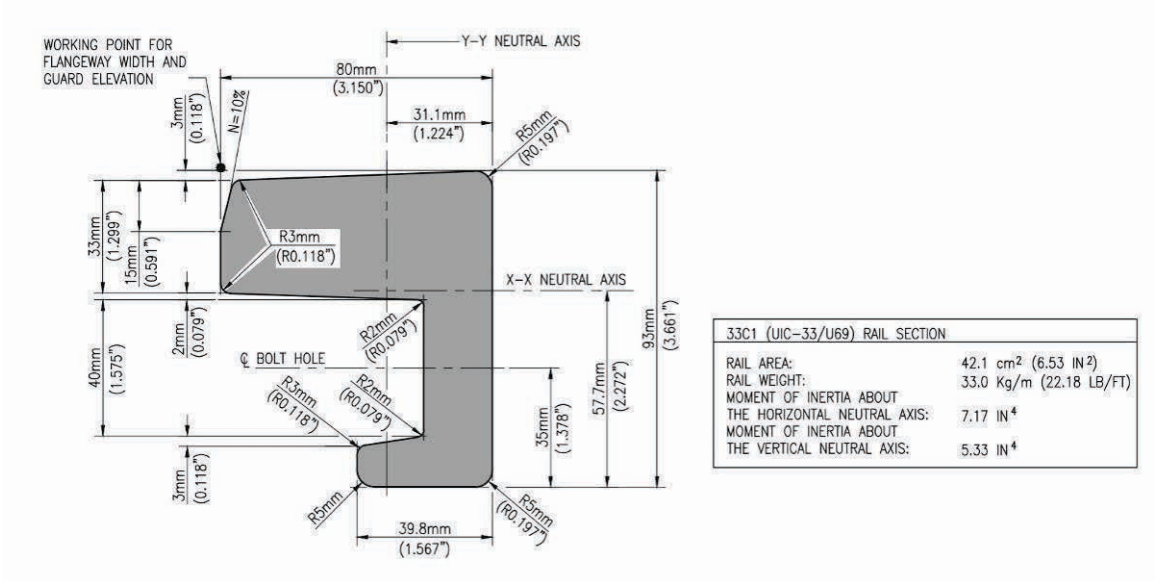


Figure 5.3.3 33C1 restraining rail

The major advantage of using the 33C1 section as a restraining rail is the capability of independent mounting from the running rail, as shown in Figure 5.3.1. In addition, the 33C1 section's independent bracket mounting assembly eliminates very nearly all of the negative issues related to field drilling of the running rails to match the restraining rail hole pattern and placement of fasteners and cross ties. Use of the 33C1 section also simplifies thermal stress adjustment of the running rail because it is structurally independent of the restraining rail.

To improve on its function as a restraining rail, the 33C1 section is typically raised above the plane of the running rails. In a common design, the working face of the guard is positioned ¾ inch [19 millimeters] above the top of the running rail to intercept an appreciable amount of the back face of the wheel. See Chapter 4, Article 4.3.5.2, for additional discussion on restraining rail height.

The independent mounting is provided by a mounting bracket that allows the restraining rail to be bolt mounted adjacent to the running rail, providing the required adjustable flangeway width. Bracket designs have evolved to the use of elastic spring clips to secure the 33C1 section in place, eliminating the need to drill the 33C1 section. The mounting design of the bracket can either be separate from the running rail fastening plate or direct fixation fastener or an integral part of the fastening plate.

Even though 33C1 restraining rail isn't physically attached to the running rail, broken running rail protection can be compromised if the mounting bracket for the 33C1 is on a baseplate shared with the running rail. If this single plate design is used and track circuits are used for broken rail detection, a detail will be required to insulate the restraining rail from the brackets. The structural loadings at the connection between the 33C1 and the bracket are significant and may be beyond the capacity of some insulating materials.

While 33C1 is sometimes employed as a frog guard rail in embedded special trackwork, it is not recommended for use as a continuous restraining rail in embedded track as it would be very difficult to insulate the bracket assemblies from the embedding pavement. Other designs of restraining rail, such as vertically mounted tee rail, are much easier to insulate and hence would be better for this purpose.

Early installations of 33C1 restraining rail required drilling it for a bolted attachment to the supporting bracket. This drilling needed to be done in the field so as to exactly match the as-built locations of the supporting brackets. Later designs use an elastic rail clip to hold the 33C1 section in the bracket, thereby creating a boltless assembly that does not require drilling. This greatly simplifies and speeds installation.

The 33C1 restraining rail assembly provides for flangeway width adjustment by adding shims directly behind the 33C1 restraining rail. This adjustment can be undertaken without disturbing the running rail installation.

33C1 restraining rail is customarily provided from European rolling mills in 15- and 18-meter [49.2- and 59.1-foot] lengths. By special order it can be obtained in lengths to suit North American requirements. The major restriction is handling the lengths on ships when placing the lengths below deck or into shipping containers.

Special four-bolt joint bar assemblies and insulated joint assemblies are used to join lengths of 33C1 rail. These joints are preferably located between supporting brackets so as to avoid the need for a special bracket. Insulated joints are required in the 33C1 section opposite running rail insulated joints. To allow for minor thermal movement in the 33C1 section, it is recommended that slotted holes be made in the joint bars, excluding the insulated joints.

On aerial structure installations, where thermal expansion of the structure must be accommodated, the 33C1 restraining rail mounting bolt holes at each mounting bracket should be slotted to allow the structure with the mounting bracket to move longitudinally. In boltless 33C1 mountings, thermal expansion will typically be handled through ordinary slippage beneath the toe of the elastic rail clip.

Because its asymmetrical shape causes it to curl when sprung into alignment, precurving of the 33C1 section is preferred on sharp radius curved track installations. Detailed shop drawings showing the layout of each segment of curved track, each individual restraining rail segment, and each supporting chair is required.

5.3.3 Restraining Rail Recommendations

As a guideline, the following restraining rail sections and mounting details are suggested:

- Concrete Cross Tie Track—a separate 33C1 mounting is provided by two additional anchor bolt inserts that are cast in the concrete cross tie during production. The bracket installation should be insulated from the cross tie, and the bracket should be designed to clear the running rail fastening system. These restraining rail cross ties will require different molds than those used for standard cross ties.
- Timber Cross Tie Track—there are two alternatives:
  - 33C1 mounted with the running rail on a single fastening plate. A welded assembly or cast steel fastening plate can be used. The single unit fastening plate with a bracket provides improved holding by using the weight of the vehicle to retain the plate bracket position and avoids possible displacement of the restraining rail relative to the running rail as the timber ties age and decay. The 33C1 bracket is designed to clear the running rail fastenings. If required for stray current control, the installation should be insulated at the top surface of the cross ties. If broken rail protection is required, this design requires insulating the 33C1 section from the mounting bracket, a major load transfer connection that could be detrimental to insulating components.
  - Vertically mounted tee restraining rail bolted to the running rail with the two on a single fastening plate. If required for stray current control, the installation should be insulated from the cross ties. Broken rail protection is not practical with this design.
- Direct Fixation Track—a separate 33C1 mounting is provided by two additional anchor bolt inserts cast in the direct fixation concrete plinth during plinth installation. The bracket installation should be insulated from the concrete plinth and the bracket should be designed to clear the direct fixation fastener body and components.

- Embedded Track—there are three options, listed below in order of preference. In all cases, the assembly should be insulated from the embedding pavement structure unless it is contained within an insulated bathtub. The three options are
  - A groove rail section with a flangeway of appropriate width and a guard/tram that both has sufficient thickness for the anticipated service and is at an appropriate elevation relative to the running surface.
  - Strap guard rail, provided the flangeway shape is appropriate for the wheel profile and gauge at the curve radii where guarding is desired.
  - Vertically mounted tee restraining rail bolted to the running rail, with the flangeway filled to within 2 inches [50 mm] of the top of rail.

5.3.4 Restraining Rail Thermal Expansion and Contraction

Restraining rails undergo thermal expansion and contraction just as running rails do. However, they should not be continuously welded because it would be virtually impossible to install them at the same zero thermal stress temperature as the adjacent running rails. It is customary, therefore, to fabricate restraining rail in segments with lengths of 30 and 39 foot (9 and 12 meters) and provide expansion gaps at bolted restraining rail joints. If the restraining rail is bolted to the adjoining running rail (such as with strap guard or vertically mounted tee rail) and the running rail is continuously welded, any connections between the restraining rail and the running rails should allow for some longitudinal movement between the two rails. This can be accomplished by drilling oversized bolt holes in the restraining rail.

Restraining rail on aerial structures may require special details when passing over expansion joints in the bridge deck.

5.3.5 Restraining Rail Restrictions

The presence of restraining rail of whatever design can complicate some track maintenance activities. Factors that should be considered include the following:

- Restraining rail interferes with the ability to orient the grinding stones of production rail grinding equipment so as to achieve optimal contact with the gauge corner of the rail. Special rail grinding equipment (typically using smaller diameter stones) and slower grinding operations will be necessary, resulting in higher rail grinding costs.

- In all open trackforms, the restraining rail can interfere with inspection and maintenance of the rail fastening system on the running rail.

- In ballasted track, some designs of restraining rail will interfere with ballast tamping operations. All restraining rail designs will trap stones during ballast dumping operations, and manual methods are usually required to remove them.

- In embedded trackforms, restraining rails increase the amount of metal on the surface of the pavement. Such steel surfaces can be slippery when wet, and there may be resulting safety issues for motorists and pedestrians.

5.4 RAIL FASTENINGS AND FASTENERS

Perhaps the most important elements of the track assembly are the devices that hold the rails to proper alignment and gauge. These items are called rail fasteners and fastenings. While these terms are often used interchangeably, they are actually distinct items. Therefore, before proceeding further, it is appropriate to define those terms as well as two other related terms that will be mentioned in this discussion.

5.4.1 Definitions

Four important terms related to track assembly—fastenings, fasteners, elastic, and resilient— are defined below:

- Fastenings—the miscellaneous hardware used to secure the rail to an underlying structural base thereby controlling rail uplift, lateral movement, and longitudinal movement. Examples of fastenings are elastic rail clips, rigid rail clips, and, the case of timber tie ballasted track, ordinary track spikes and rail anchors. In the case of elastic and rigid rail clips, the fastenings may also include insulating pads to electrically isolate the rail from grounded items below the base of rail.
- Fasteners—plate assemblies that incorporate both the rail fastenings described above and a system to anchor the plate to an underlying base. Depending on the product design, the electrical isolation requirements can be achieved in the rail fastenings (as described above), within the fastener plate, or beneath the fastener plate.
- Elastic—an adjective typically applied to a rail fastening device made of spring steel and used to hold a rail into a rail seat on either a rail fastener or a cross tie. Elastic rail clips are often called “spring clips” or simply “rail clips.”
- Resilient—an adjective typically applied to a rail fastener assembly that incorporates elastomeric elements used to dampen vibrations that originate at the rail/wheel interface and thereby minimize their transmission to locations outside of the track structure.

Because of the large number of rail fastening/fastener designs, the definitions above are somewhat imprecise, but they should suffice for the purposes of the discussion that follows. It is perhaps notable that one common railway engineering reference source reversed the definitions of “elastic” and “resilient” above.

5.4.2 An Introduction to Common Designs

Common rail fastener assemblies include the following:

- Conventional rolled steel tie plates with shoulders, punched with square holes to accept a series of cut spikes to secure the tie plate to a timber cross tie and to hold the rail in the rail seat area of the tie plate. These assemblies do not include any specific electrical isolation measures.
- Rolled, forged, or fabricated steel tie plates with shoulders that accept an elastic rail fastening and are drilled, punched, or otherwise machined to accept hold-down spikes (typically screw spikes) for attachment of the plate to a timber cross tie. These

assemblies can incorporate electrical isolation measures either at the base of rail or between the plate and the cross tie. Modifications of this rail fastener assembly are sometimes used on concrete cross ties within special trackwork.

- Manufactured plates that include an integral elastomeric pad beneath the plate that provides both electrical isolation and acoustic attenuation. The elastomeric pad may or may not be bonded to the plate during molding. If the pad is bonded to the plate, it is usually also bonded to a second plate positioned beneath the elastomeric pad. If bonded to both plates, the pad itself is typically not a premolded unit but is instead formed by injecting the molten elastomeric compound between the plates in a mold. The rail fastening assembly can be either a rigid clip held in place with a bolt assembly or an elastic clip. Two or more threaded bolts are used to anchor the plate assembly to female anchor inserts in an underlying reinforced concrete base. The overall assembly is commonly named a "direct fixation rail fastener."

As in the case of the definitions above, rail fasteners come in a wide array of designs, many of which cannot be neatly categorized. In many cases, individual components or whole assemblies may be the proprietary products of one manufacturer. Many, if not most, rail fasteners incorporate subassemblies from multiple manufacturers.

5.4.3 Insulated Fastenings and Fasteners

As noted previously, the rails are the negative return path for the traction power current to the traction power substation (TPSS). The negative return current must, to the maximum degree possible, be confined to the rail so as to control stray current leakage, which causes corrosion not only of transit tracks and structures but also nearby underground utilities and structures. For additional information on stray current protection, refer to Chapter 8.

Conventional ballasted track often relies on timber ties to insulate rails from the ground. Timber ties can actually provide a reasonably good level of rail-to-earth resistance provided the entire track structure, including the ballast, is and remains clean and dry. Maintaining those conditions, particularly in the vicinity of highway grade crossings, is particularly difficult. Therefore, although wood is considered a non-conductive material, timber cross ties will generally not provide sufficient insulation to totally isolate the track from ground over the long term. For this reason, special designs of rail fasteners and fastenings are used to isolate the rail from the cross tie. These systems will be described below.

Concrete cross ties typically use elastic rail fastening clips. The rail clips are insulated from both the rail base and the shoulders embedded in the concrete cross tie by plastic insulators. The rail is further insulated in the concrete cross tie's rail seat area by a pad that is used primarily to prevent the rail from abrading the concrete but that also provides electrical isolation. These pads are often dimpled or ridged with shape factors so as to provide resiliency.

Direct fixation track typically uses a direct fixation fastener plate system consisting of a fastener body that incorporates elastomeric elements to provide resiliency and electrical isolation, a rail fastening system to hold the rail to the fastener body's rail seat area, and an anchorage assembly to secure the fastener body to a substrate—usually a concrete slab. The rail fastening system is

typically elastic rail clips, although rigid clips held in place by a bolt assembly have been used. The anchorage assembly typically consists of two or more bolts that pass through the fastener body and are then threaded into female anchor inserts embedded in the underlying track slab or plinth.

Direct fixation rail fasteners will occasionally be used on timber or concrete cross ties where additional acoustic attenuation is desired. They have also occasionally been used in specialized embedded track installations, but this is uncommon.

Most embedded track systems do not include rail fasteners, but most do include some sort of rail fastenings, most often to attach booted rail to an embedded steel (or plastic) cross tie or a steel leveling beam.

5.4.3.1 Isolation at the Rail Base

To provide electrical isolation in open track forms, the rail must be isolated from the surrounding track components. The simplest and most reliable location for this isolation is at the surfaces of the rail. Insulators are placed between the base of the rail and the underlying mounting surface and also between the base of rail and the rail clip assemblies, as shown in **Figure 5.4.1**.

5.4.3.2 Isolation at the Fastener Base

To provide electrical isolation of the fastener from the surrounding track components, the insulating barrier must be installed at the base of the fastener plate or mounting surface. The insulating barrier consists of an insulated base fastener pad and insulating flanged bushings (also known as thimble collars) surrounding the anchoring screws or bolts, as shown in **Figure 5.4.2**.

5.4.4 Elastic Rail Clips

Elastic (spring) clips come in many designs, almost all of them proprietary. Examples are various designs of clips made by Pandrol (e.g., the “e-clip,” the “Fast Clip,” and the original PR-series clips), the McKay/Safelock clip, and clips made by Vossloh, to name only a few. All of these clips have merits, and the selection of one versus the other is largely a matter of personal choice. In general, it is preferable to minimize the number of rail clip styles on a transit property so as to simplify maintenance inventory.

Some designs of rail clips are installed by inserting them into a shoulder in a direction parallel to the rail while others are installed perpendicular to the rail. In rare instances, one make and model of clip might therefore be preferable to another based on physical space constraints, as often occurs within special trackwork.

The toe loads applied by spring clips can be varied, usually through modification of the shoulders into which they are inserted. As manufactured items, the actual toe load applied to the rail base is subject to some tolerances and will vary at installation. A spring clip’s toe load will also slacken off due to creep with years of service and/or multiple removals and reapplications. There are also various designs of rail clips that apply reduced or even zero toe load so as to allow controlled longitudinal movement of the rail. In some cases, these special clips are visually nearly identical to the standard clip and care must be taken to ensure the correct model of clip is installed at a given location.

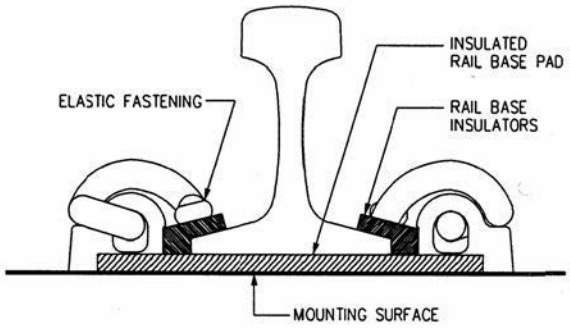


Figure 5.4.1 Isolation at the rail base

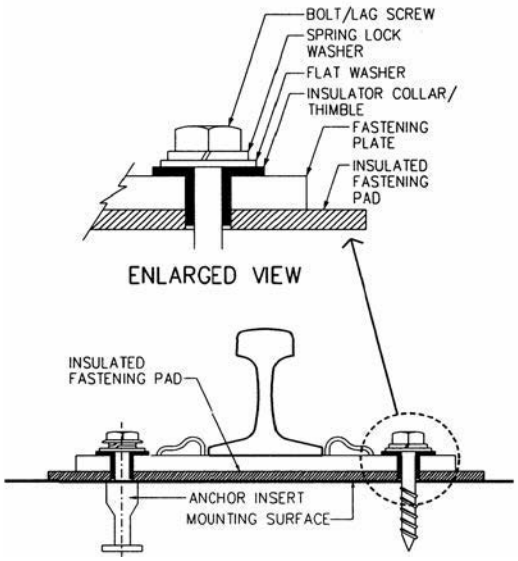


Figure 5.4.2 Isolation at the fastener base

The steel in elastic rail clips starts as straight bar stock but is then hot-worked into the shape of the finished clip. The clips undergo significant stress during manufacture and are further cold stressed when inserted into the rail fastener shoulder and thereby stretched so as to apply force to the base of rail. Steel that is in a state of tensile stress will corrode more easily because, at an atomic level, there is more space within the material for hydrogen to enter and cause embrittlement. Steel spring clips that are placed in a corrosive environment (such as a damp subway tunnel) or that are routinely exposed to chemicals (such as salt brine from roadway de-icing products) can corrode and fail.

The worst conditions can occur when both of those conditions occur underground, since there is no opportunity for natural precipitation to wash contaminants off the track. One transit system with such an environment experienced failure of a double-digit percentage of its elastic rail clips within 2 years of the installation of new direct fixation rail fasteners. The problem was compounded when it was discovered that the stumps of the broken-off spring clips could not be

extracted from the rail fastener shoulders because exfoliated rust had filled the small void space around the shank of the rail clips, trapping them in place.

Because of such problems, it is strongly recommended that if spring clips are proposed for use in such a hostile environment, consideration should be given to specifying that the clips have a robust and proven protective coating. One clip manufacturer has achieved a measure of success in a very corrosive environment with a zinc silicate coating topped by a second coating of paraffin wax.

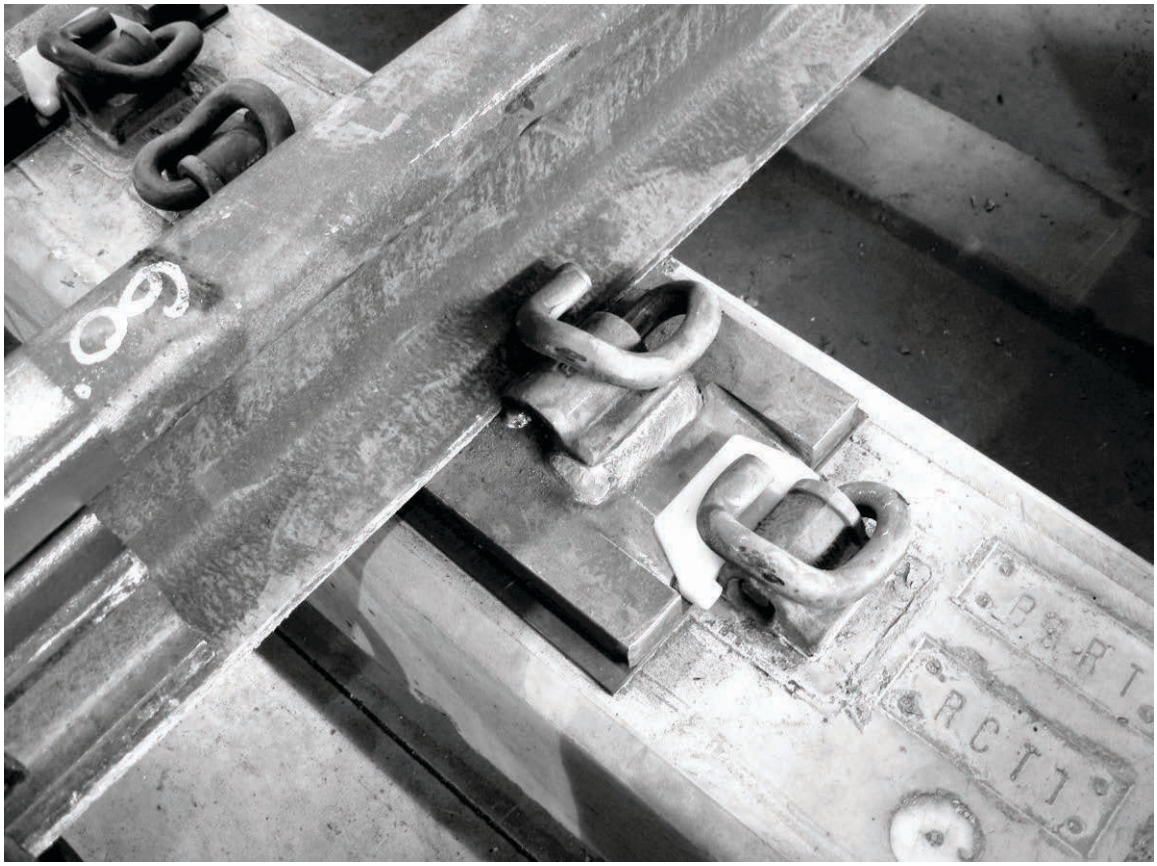
Some designs of elastic rail clips were formerly proprietary products and thus available from only one source; however, now these designs are either in the public domain or are made by multiple vendors under license. For example, the "e-clip," which was originally the proprietary product of a firm in the United Kingdom, is now offered by many manufacturers worldwide. Caution is required when procuring such items because, while they may be dimensionally identical to the original product, their material composition and the quality procedures under which they are manufactured could be appreciably different.

5.4.5 Fastenings for Timber and Concrete Cross Ties for Ballasted Track

The original rail fastening system for track constructed on timber cross ties in the traditional manner included rolled steel tie plates, cut spikes, and rail anchors. The latter devices clamp to the base of the rails, bear against the sides of the cross ties, and are provided to restrain the rail from longitudinal movement. Since the rail anchors project down well below the elevation of both the rail and the tie plate, it is inevitable that they will be in contact with the ballast. Unless the ballast is absolutely clean and dry, this creates a direct path for stray currents from the rail to ground. Insulating these rail anchors is virtually impossible since any conceivable coating system would be subject to abrasion damage, allowing current leakage. For this reason alone, conventional rail anchors are not recommended for LRT use.

Elastic spring clip rail fastening systems, which have no projection down into the ballast section (and also provide superior longitudinal restraint for continuously welded rail) are recommended for any LRT timber tie ballasted track.

Main line transit track with timber cross ties must utilize an insulation method similar to that shown in Figure 5.4.2, with screw spikes used to secure the tie plate. For simplicity, Figure 5.4.2 illustrates rail with no rail cant; if cant is required, rolled plates manufactured with the desired cant can be used. In special circumstances, where an off-the-shelf rolled plate will not work, the rail seat area of the plate can be milled to provide the desired cant. Special trackwork installations, whether on timber or concrete cross ties, usually require a plate insulation system similar to that shown in Figure 5.4.2. Such designs have a proven service record. An alternative method of anchoring large plates to concrete switch ties embeds a shoulder in the tie that either projects through a large hole in the plate or is adjacent to the end of the plate. Insulators and an elastic rail clip are then used to secure the plate to the shoulder and the tie, as shown in **Figure 5.4.3**.



(Photo courtesy of Bay Area Rapid Transit)

Figure 5.4.3 Threadless plate fastenings on concrete switch ties

The elastomer pad in direct fixation rail fasteners has been manufactured from natural rubber, synthetic elastomers, and polyurethane products. These materials have been formulated to provide both high- and low-spring rates for the track as well as sufficient electrical isolation. The resilient elastomeric components come in widely varying configurations, grades, and spring rates.

Early designs of direct fixation fasteners often specified an anchorage system consisting of an embedded stud bolt projecting up out of the concrete invert and through the rail fastener body, capped by spring washers and nuts. While this simplified some steps during construction, it complicated other construction activities as well as maintenance. For example, to remove or change out a rail fastener, the rail had to be lifted very high to allow the fastener to clear the projecting stud bolt height. To shim the fastener to adjust the rail height, slotted shims were designed with slots for easy installation around the projecting bolts, but such shims were worked out from beneath the fastener under traffic, resulting in a loose fastener installation. For these reasons, stud bolts are not recommended.

The preferred design for anchoring direct fixation rail fasteners to the underlying concrete slab or plinth consists of bolting through the rail fastener base plate to embedded female anchor inserts. With female anchor inserts, the shim need only include holes matching the pattern of the

fastener's anchor inserts. Shimming of the fastener requires only removing the anchor bolts and lifting the rail and fastener body a small amount more than the desired shim thickness.

Early direct fixation fastener designs incorporated anchor bolt assemblies that passed through both the top and bottom plates of the fastener body. Modern designs configure the body so that the anchor bolts pass only through the bottom plate. This approach eliminates bending moments in the anchor bolts due to lateral forces applied to the rails by the wheels, but requires that the fastener body incorporate other measures to keep the top plate aligned with the bottom plate.

Most direct fixation rail fasteners include some degree of lateral adjustment capability in the anchor assemblies. This is provided for the purpose of adjusting gauge to within tolerances during construction and corrective maintenance. The adjustment capability obviously adds some cost to the fastener assembly and is also a potential weak spot in the design. Moreover, if the top-down construction method is used (see Chapters 4 and 13 for further discussion of top-down construction), it is entirely practical to build the track without needing to rely on lateral adjustment. It has also been rather uncommon for any transit agency maintenance organization to subsequently use this lateral adjustment for any purpose. For these reasons, some transit agencies have opted for direct fixation rail fastener designs that provide no lateral adjustment at either the anchor bolt or anywhere else in the fastener. One transit agency reasons that lateral adjustment capacity merely gives the constructor an excuse for doing shoddy work, knowing such work can be "corrected" through the use of lateral adjustment. Omitting the lateral adjustment feature forces the contractor to "do it right the first time," resulting in an arguably superior end product.

Nearly all of these direct fixation rail fastener designs are proprietary products of a single manufacturer. For all practical purposes, there are no "standard" designs in the public domain. As a result, for any project that is being constructed with public funding, it is typically necessary for the track designer to first make certain decisions about the general form of the direct fixation track and then prepare specifications that specify the required performance characteristics (but not the details) of the direct fixation rail fasteners. In the case of small procurements for short lengths of direct fixation track, it is sometimes possible to specify a fastener by make and model number "or equal"; however, the product procured may not be tuned to the noise and vibration attenuation needs of the project.

5.4.6 Fasteners for Direct Fixation Track

5.4.6.1 Fastener Design Consideration

TCRP Project D-5 and TCRP Project D-7, Task 11, performed extensive investigations into direct fixation track systems. Those investigations resulted in *TCRP Report 71: Track-Related Research—Volume 6: Direct-Fixation Track Design Specifications, Research, and Related Material*, a comprehensive two-part treatise on direct fixation track that includes *CRP-CD-61* with useful software applications for design of direct fixation track systems.

This Handbook will not attempt to duplicate the contents of *TCRP Report 71*, *Volume 6*. However, as a primer to the topic, the following discussions of some of the principal mechanical characteristics of direct fixation rail fasteners are offered. Readers of this Handbook are strongly encouraged to obtain copies of *TCRP Report 71*, *Volume 6*, prior to undertaking direct fixation track design.

5.4.6.1.1 Vertical Static Stiffness

Vertical static stiffness is often called spring rate, and represents the slope of the load versus deflection over a prescribed range of 1,000 to 12,000 pounds (5,000 to 55,000 N). Current light rail track designs include a static stiffness of about 100,000 to 120,000 pounds per inch [18 to 21 MN/m], which, with a 30-inch [760-millimeter] fastener spacing, gives a rail support modulus of about 3,700 pounds per square inch [26 MN/m^2]. One feature of low-stiffness fasteners is that they distribute rail static deflection over a larger number of fasteners, making the rail appear more uniformly supported. Low rail support stiffness reduces the pinned-pinned mode resonance frequency due to discrete rail supports, as well as the rail-on-fastener vertical resonance frequency. Static stiffness in the 100,000 to 120,000 pounds per inch [18 to 21 NM/m] range provides reasonable control of track deflection in the vertical direction without unduly compromising lateral stiffness. Noise and vibration experts in the field have additional thoughts on controlling the pinned-pinned mode resonance by varying the fastener spacings. For additional information on this subject, refer to Chapter 9, Article 9.2.1.2.2.

The typical static stiffness of direct fixation fasteners used by various U.S. systems is on the order of 112,000 to 280,000 pounds per inch [20 to 50 MN/m], with spacing ranging from about 24 to 30 inches [610 to 762 millimeters]. At least one U.S. rail transit property uses a 36-inch [910-mm] fastener spacing; however, as is explained in Chapter 9 of this Handbook, wide spacings can have undesirable harmonic characteristics, potentially exacerbating the tendency to develop rail corrugations. So as to both deter corrugation growth and better approximate the stiffness of ballasted track, softer fasteners have been developed with an elastomer stiffness on the order of 106,000 pounds per inch [18.6 kN/m]. These fasteners incorporate elastomer bonded between a ductile iron or steel top plate and stamped steel base plate. A snubber is installed between the top and bottom plates, beneath the rail seat, to limit lateral motion of the top plate. Lateral rail head stiffness is on the order of 30,000 pounds per inch [5 MN/m].

Fasteners have been supplied with vertical stiffness on the order of 110,000 pounds per inch [19 MN/m], but with very low lateral stiffness, on the order of 9,800 pounds per inch [1.75 MN/m], due to lack of a snubber or other lateral restraint. These differences in lateral stiffness reflect differences in design philosophy concerning maximum allowable gauge widening under load and consequent sharing of lateral load between fasteners in curved track.

5.4.6.1.2 Ratio of Dynamic to Static Stiffness (Vertical)

The ratio of vertical dynamic to static stiffness is a very important quantity that describes the quality of the elastomer. A low ratio is desirable to maintain a high degree of vibration isolation. A desirable upper limit on the ratio is 1.4, which is easily obtained with fasteners manufactured with a natural rubber elastomer or a rubber derivative. Ratios of 1.3 are not uncommon with natural rubber elastomer in shear designs. As a rule, elastomers capable of meeting the limit of 1.4 must be of high quality and generally exhibit low creep.

5.4.6.1.3 Lateral Restraint

Lateral restraint is the ability of the fastener to horizontally restrain the rail. High lateral restraint is often incompatible with vibration isolation design requirements. Therefore, fasteners that provide adequate stiffness to guarantee both an adequate degree of horizontal position control as well as vibration isolation are desirable. Snubbers are protruding portions of metal plate that penetrate the adjoining plate to act as a limit flange in controlling lateral displacement. Some

fasteners use an upsweep curved bottom plate design to restrain or act as the limiting flange. The guiding design principle is to provide a three degree-of-freedom isolator. Hard snubbers are undesirable in fasteners, because they limit vibration isolation only in the vertical direction.

5.4.6.1.4 Lateral Stiffness at the Rail Head

Lateral stiffness is measured at the rail head and includes the effect of fastener top-plate rotation. For track to stay in gauge, the rail fasteners must maintain rail head position within tolerances on both curves and tangent track. This goal is potentially in conflict with the requirement for horizontal vibration isolation. The lateral deflection of the top plate of typical sandwich fasteners is limited by either the snubbers or the configuration of the bottom plate and to a lesser extent by the elastomer in shear. If the snubber is located beneath the rail, a low fastener with low vertical stiffness will have low rotational stiffness and thus poor rail head control. This conflict has been overcome by one European design, which incorporates elastomer in shear with a large lateral dimension to resist overturning. Another way of overcoming this potential conflict is to move most of the elastomer to the ends of the fastener, away from the rail center, thus maximizing the reaction moment to overturning forces. If a snubber is located towards the lateral ends of the fastener, it will minimize rotation of the rail by forcing the rail to rotate about a point located towards the field side of the rail in response to gauge face forces.

Designers should not confuse the amount of rail head deflection that occurs during laboratory testing with actual gauge widening under load in service. Typical laboratory testing occurs with only one or two rail fasteners beneath a test rail. Track in service does not see loading of a single fastener. Instead, the rail spreads the load so as to be shared by three, four or even more fasteners. A fastener that is laterally soft will result in more load sharing between fasteners, reducing the load experienced by each, and yet still result in acceptable track gauge control. The torsional stiffness of the rail also is a factor in gauge control.

5.4.6.2 Shims beneath Direct Fixation Rail Fasteners

The construction of the concrete plinths or grout pads for support of direct fixation rail fasteners is, like all concrete work, subject to some construction tolerances. Particularly in the case of “bottom-up” construction of direct fixation track, it can be difficult to get the plinth concrete at precisely the correct elevation. If the bearing surfaces beneath the rail fasteners are not at a uniform distance below the top-of-rail profile, some fasteners may be stretched vertically while others are in compression, hence taking a disproportionate amount of the vertical loading. So as to avoid this condition, shims are usually selectively inserted beneath the direct fixation rail fastener body. A typical criterion is that if a taper gauge shows a gap of 1/16 inch [1.5 mm] or more between the base of the unclipped rail and the top plate of the rail fastener, the fastener should be shimmed.

Shims have been made from a variety of materials, but galvanized steel and high density polyethelene (HDPE) are the most common. Shims are typically made so as to project ¼ to ½ inch [6 to 12 mm] beyond the footprint of the rail fastener body. The HDPE shims will, in theory, provide an additional electrical barrier between the rail fastener and ground and, if they project beyond the footprint of the fastener bottom plate, will increase the surface leakage distance, at least when clean. The shim obviously must have holes to allow the anchor bolts to pass through. Some contractors propose shims with slotted holes that extend out to the edges of the shim, thereby making it possible to insert the shim without fully removing the anchor bolts. However,

experience has shown that slotted shims can shift out of position during service, and they are therefore not recommended.

Shims are typically provided in a variety of thicknesses, and the contractor will stack shims of one or more thicknesses to achieve the requisite shimmed height. However, both the number of shims and the total shimmed height beneath any rail fastener should be limited. There are two reasons for this:

- The fastener is most stable if it is placed directly on the concrete plinth with no shims. Each shim introduces an intermediate shear plane where slippage could occur.

- Excessively shimming the fastener can alter the way in which the anchor bolts are loaded, including the amount of thread engagement and the moment loading seen at the root of the threads at the plinth surface.

For these reasons, it is recommended that not more than three shims be inserted under any direct fixation rail fastener and that the total shimmed height be not more than about ⅜ inch [9.5 mm]. Higher shimmed heights have been used, particularly on "plinthless" design direct fixation aerial structures, but any such installations should be carefully analyzed.

Extremely thin shims are subject to failure. For this reason, many designs specify that every rail fastener should have a shim of the minimum acceptable thickness—usually ⅛ inch [3 mm]. Then, if additional shimming is required, that shim is removed and a shim or shims of the required total thickness are inserted.

See Chapter 13, Article 13.3.2, for additional discussion on shims in direct fixation track.

5.4.7 Fasteners and Fastenings for Embedded Track

Embedded tracks typically do not use a rail fastener as the term is defined above. Some embedded tracks—those where the rail is "floating" in a trough filled with an elastomeric grout—use no rail fastenings at all. More common is the use of a some sort of clip to hold a booted rail to either a steel tie/leveling beam or an individual plate fastener.

Booted rail in embedded track can use a non-insulated rail assembly; however, a protective insulator is typically employed beneath the clip to spread the hold-down load and avoid damage to the rail boot. Elastic rail clip assemblies very similar to those used on concrete cross ties are a common choice. The use of an elastic spring clip in embedded track is usually based on an expectation that it will eventually be necessary to change out rails—either on a spot basis or out-of-face—at some time prior to when the embedding pavement structure is life expired. Elastic rail clips greatly facilitate rail change out in ordinary open ballasted track, and it is therefore presumed that similar benefits would ensue from using them in embedded track as well.

So as to ensure that the elastic clips remain limber and are not encased in the concrete pavement, they are usually covered with a plastic cap commonly called a "batter's helmet." **Figure 5.4.4** illustrates a typical installation of this type. Items to note include the heavy duty pad beneath the toe of the elastic clip, so as to protect the rail boot from abrasion, the helmet ready

for installation, and the jacking screws on the end of the leveling beam. (The track has not yet been elevated to final grade in this view.)

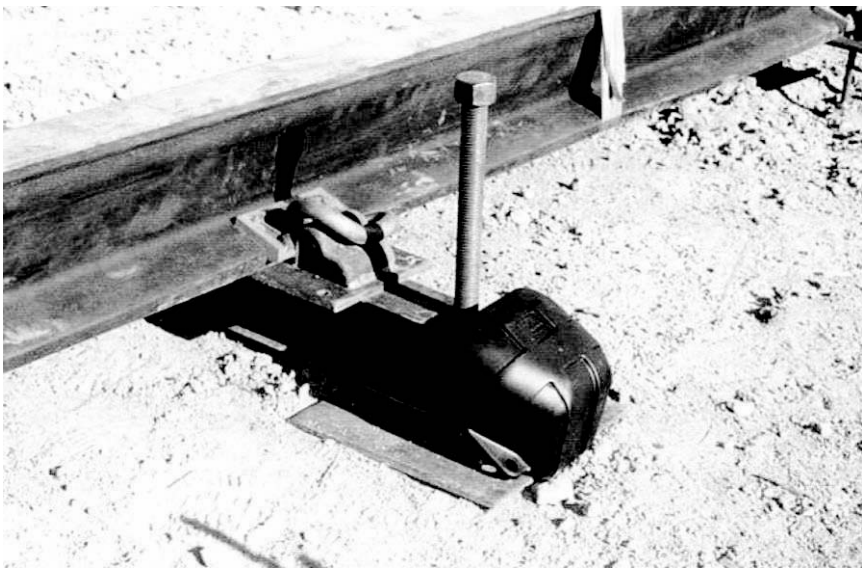


Figure 5.4.4 Elastic rail clip assembly for embedded track

Potential problems with use of elastic clips in embedded track include the following:

- Since the elastic clip is installed by hammering it into a shoulder, there is the possibility of a misaimed hammer swing damaging the rail boot. If the damage is not noted and immediately patched, a stray current leak might result.

- The intentional void beneath the helmet and around the elastic rail clip is not watertight. There is a reasonably good chance that it will periodically be at least partially filled with water, beginning with bleed water from the fresh embedding concrete and later including surface water that leaks through small cracks in the pavement surface. This moisture could initiate corrosion and result in clip failure. (Note also the discussion in Article 5.4.4 on how elastic clips can be subject to brittle fracture when in a corrosive environment.) Since there is no way to inspect, much less easily replace, the clips once the pavement concrete has been poured, loss of rail anchorage might not be apparent until successive failures of adjacent clips have left the rail sufficiently loose to visibly move under traffic.

Because of these issues, many designers choose to use a rigid clamp style clip to hold the booted rail to the steel leveling beam. Such clips can be obtained either as steel fabrications, steel or iron castings, or in molded plastic. Hold-down clips should have a smooth finish with rounded edges and corners so as to avoid damage to the boot. All the edges and corners should have ample radii as the clips may rotate slightly when mounting bolts are torqued.

If the embedding material includes a substantial amount of reinforcing steel above the base of rail elevation, the pavement itself will serve to hold the rails to alignment and gauge. Under such circumstances, the rail hold-down clips and leveling beams effectively have only one function—to hold the booted rail in place until the embedment concrete is installed. Under such

circumstances, the use of plastic rail clips can reasonably be considered. However, should the embedment material be of a nature to not securely restrain the booted rail (as in the case of turf track), then the rail hold-down clip design must be sufficiently robust to hold the rail in alignment without any other supports. The use of plastic rail clips is not recommended under such circumstances.

As a guideline, steel rail clips are recommended for any installation where the rail fastening assembly can be expected to take some measurable amount of either lateral load or uplift/rollover load. Plastic clips can be used in circumstances where rail loadings are light or when the reinforcing steel in the embedding concrete is sufficiently robust that the concrete pavement acts as a supplemental, if not the primary, means of holding the rails in position.

5.5 CROSS TIES AND SWITCH TIES

Ballasted track requires cross ties to support the rail. Chapter 4 discusses cross ties in the design of ballasted track. Cross ties are used mainly for ballasted track, although they are occasionally used in direct fixation encased track, where a cross tie or sections thereof are partially encased in a concrete track structure, and in embedded track, where the cross tie is fully encased in the track structure.

Cross ties are generally made of three materials: timber, concrete, and steel. In addition, cross ties made of plastic and composites of plastics and other materials are now available.

Several designs of plastic and composite material cross ties are on the market, but all are proprietary products and none have seen wide acceptance. Handbook users who are interested in such materials should consult industry trade magazines and vendors for current information.

Light rail transit systems use both timber and concrete cross ties in ballasted track. However, current designs of prestressed precast concrete cross ties, which feature embedded shoulders for rail clip assemblies plus sundry inserts for the application of trackwork components, are available at first cost prices that are very competitive with insulated timber ties. When life cycle costs are considered, concrete ties are nearly always a better choice and have therefore become the preferred product on most rail transit systems.

5.5.1 Timber Cross Ties

The timber species currently used in cross ties varies somewhat by geography, but includes selected hardwoods with occasional consideration of tropical hardwood species. The expense associated with cross tie replacement (including the intangible costs associated with disruption of revenue transit operations) makes it highly desirable to use high-quality timber ties during initial construction. Species of timber commonly used in freight railroad track construction in the region of the project are generally satisfactory. As a guideline, timber cross ties for light rail transit use should be hardwood—preferably oak in the eastern United States and Douglas fir in the western United States.

There are risks and rewards associated with the use of tropical hardwoods (e.g., Azobe). Designers must research the topic thoroughly. It is very important to specify such exotic woods

by their botanical name. The use of a common name can result in obtaining an inferior material. For example, there are several species of Azobe with wildly different performance characteristics. The problem is that they are virtually indistinguishable from each other after the bark has been removed. On-site inspection at the sawmill is therefore essential. Certain grades of Azobe have been known to develop a fungus and decay after only a short period of time under certain track conditions. All things considered, the use of either high-quality domestic timber or concrete cross ties may be preferable to the use of imported hardwoods.

The requirement for an insulated tie plate to be mounted on a timber cross tie dictates the general width of the cross tie. Standard tie plate widths range from 7 to 7 ½ inches [180 to 190 millimeters]. An insulated tie pad protrudes a minimum of a ½ inch [12 millimeters] on all sides of the tie plate, which results in a minimum tie width of 8 inches [204 millimeters]. A 9-inch- [230-millimeter-] wide timber tie provides sufficient surface to support the total insulator pad with no overhang beyond the edge of the tie. Skewed tie plates at special trackwork locations should be used with consideration of the overhang issue in relation to degree of the skew angle. To overcome the skew position, specially fabricated tie plates with angled shoulders are usually required. Very often, each such plate will need to be custom made for only one location due to turnout angle and plates being right and left handed in specialwork.

Timber ties should generally be 7 x 9 inches [180 x 230 millimeters] in cross section. Per AREMA specification, the 7-inch tie depth is referred to as a “7-inch grade” cross tie. There is no equivalent metric name, as the S.I. system is not used to classify and name tie sizes. Notably, timber ties overseas (where they are usually called “sleepers”) are typically sawn to radically different dimensions than timber ties in North America. In the United Kingdom and other parts of the world influenced by British practice, a typical sleeper will be roughly 125 x 250 millimeters [about 5 x 10 inches] in cross section.

Customary lengths of cross ties stem from standard track gauge of 56.5 inches [1435 millimeters] and are generally 8 feet 6 inches [2.59 meters]. Cross tie length is generally set by considering the contact pressure between the bottom surface of the cross tie and the ballast under railroad loading. Under transit loading, the cross ties can generally be shorter and less wide, but other reasons make it more practical to conform to the railroad dimensions. For the same reasons, it is generally unnecessary to use longer cross ties for plain track on broad gauge systems. Broad gauge systems do require longer switch ties.

Timber cross ties are generally manufactured to conform to the current specifications of the AREMA *Manual for Railway Engineering*, Chapter 30—Ties, Part 3—Solid Sawn Timber Ties. AREMA defers to another professional trade group, the American Wood Preservers Association (AWPA) for many issues regarding preservative treatment of timber ties. When using timber cross ties conforming to AREMA recommendations, the type of wood, tie size, anti-splitting device, incising, wood preservative treatment, and machining should be specified in the procurement contract.

Regardless of species or preservative treatment methods, timber cross ties will eventually require replacement. In LRT service, the usual failure mode for timber ties is decay rather than mechanical wear. If the guideway configuration does not facilitate the replacement of individual cross ties, this future maintenance activity will be very costly. For example, if timber cross ties

are used in a curbed ballast section such as that shown in Chapter 4, Figure 4.5.5, it may not be possible to change out individual defective ties, particularly if the track center dimension is less than about 13 feet [4.0 meters]. Under such tight conditions, maintenance forces would need to change out clusters of ties (whether defective or not) by removing ballast, rotating the ties 90 degrees, and lifting them out from the gauge area of the track. Alternatively, the rail could be set aside, allowing full access to all ties albeit only by taking the track out of service for an extended period. Either operation would be costly. Because of such circumstances, timber cross ties are not recommended for curbed ballast sections. Either concrete cross ties or an entirely different trackform should be considered for such areas.

For light rail transit systems constructed in the early 1980s, timber cross ties alone were believed to provide sufficient electrical isolation. Current standards require a layer of insulation between the bottom of the tie plate and the top of the tie plus insulating thimbles between the lag screw and the tie plate, as shown in Figure 5.4.2.

The environmental aspects of treating the wood and ultimately disposing of old cross ties after their service life has expired has raised their life cycle costs. While the preservatives used to treat timber cross ties do not result in their being classified as hazardous waste, they do need to be disposed of in a controlled manner. This is usually in a licensed landfill, although many are burned as fuel at cogeneration facilities.

5.5.2 Concrete Cross Ties

Concrete cross ties have become very common in light rail transit designs as even first cost makes them competitive with timber cross ties with insulated tie plates. Life cycle costing of concrete versus timber ties makes the case for concrete even more compelling. The most common concrete cross tie is the pretensioned monoblock tie with embedded cast steel shoulders for an elastic rail clip assembly. The rail fastening system includes insulating rail seat pad and combination clip and shoulder insulators, as shown in Figure 5.4.1.

The typical plan view dimensions at the base of the concrete ties for rail transit are 10 inches [255 millimeters] wide and 8 feet 3 inches [2515 millimeters] long. Vertically, the center of the tie is typically tapered with a 7-½-inch [190-millimeter] height at the rail seat and a 6-½-inch [165-millimeter] height at the center of the tie.

In addition to conventional cross ties that hold only the two running rails, special cross tie designs are available to hold restraining rail in guarded track and emergency guard rails. The overall size of the various types of cross ties is similar except that the height at the center of the tie will increase to support the supplemental assemblies at the appropriate elevation. The configuration of the restraining rail and emergency guardrail cross ties provides a relatively level surface between the running rails.

The length of concrete cross ties may vary among transit systems; however, 8 feet 3 inches [2515 millimeters] appears to be the most common length for standard track gauge. Extra length cross ties are recommended for supporting the field side panels of modular at-grade highway crossing systems. Additional width in the field side crossing panels has been demonstrated to

provide better panel stability. Extra long cross ties can also be considered for transition zones between ballasted track and stiffer trackforms, as shown in Chapter 4, Figure 4.4.1.

The concrete cross tie design for light rail transit track is based on the light rail vehicle weight, anticipated loads, and vehicle operating velocity. It is generally a less robust version of the railroad concrete cross tie with less reinforcement (and sometimes a reduced cross section). Designs of concrete ties for transit application are tested for positive and negative rail seat bending and tie center bending using the procedures specified in AREMA's *Manual for Railway Engineering*, Chapter 30. Compared to concrete cross ties for railroad service, the only differences in the design and testing regimen are the vehicle loadings, cross tie spacing, and speeds that are factored into the AREMA equations. Concrete cross ties for transit use are generally specified in accordance with AREMA Chapter 30, the only difference being the vehicle loadings, cross tie spacing, and speeds that are factored into the AREMA design equations.

5.5.2.1 Concrete Cross Tie Design

The design of concrete cross ties for light rail transit track is based on performance specifications that consider the following:

- Tie spacing.
- Tie size.
- Rail section.
- Rail fastening system.
- Wheel loads.
- Impact factor.

5.5.2.2 Concrete Cross Tie Testing

Prior to acceptance of the concrete cross tie design, the manufactured cross tie should be tested for compliance with specifications and the determined calculated load limits. The tests should be conducted in accordance with the procedures outlined in the AREMA *Manual for Railway Engineering*, Chapter 30.

5.5.3 Switch Ties—Timber and Concrete

Switch ties for LRT special trackwork installations include both timber and concrete.

5.5.3.1 Timber Switch Ties

While economics are shifting in the direction of concrete switch ties for standardized installations, hardwood timber switch ties can still be an economical choice for unique special trackwork layouts.

With the exception of length and related parameters, such as straightness, the requirements for timber switch ties for LRT use are generally as discussed above for standard length timber cross ties. Timber switch ties are typically provided in lengths ranging from 9 feet [2.75 meters] to 17 feet [5.20 meters]. Timbers as long as 23 feet [7 meters] are occasionally used on crossover and double crossover tracks.

The headblock ties beneath switch machines are often specified to be 9 inches [230 millimeters] thick. This allows the tie to be dapped, thereby allowing a lowered switch machine mounting,

reducing the projection of the switch machine above top-of-rail elevation. This low profile is generally necessary only on heavy rail transit systems where additional clearance is desired because of the third rail shoes on the transit vehicle. That dimensional constraint in turn led to specialized designs of switch machines and rods for transit use. As is often the case, details developed for one mode of railway are carried over into other modes so as to take advantage of off-the-shelf hardware, hence the use of dapped switch ties on many LRT projects.

Similar to main line timber cross ties, the spatial requirements for insulated rail fastener plates often dictates the width of the tie. A 9-inch- [230-millimeter-] wide timber switch tie usually provides adequate surface to support the entire insulator pad with no overhang beyond the edge of the tie. Customized plate designs are usually needed so that the plate can be mounted parallel to, and entirely on, the tie surface. Neither the plates nor the underlying insulating pads should project beyond the edges of tie.

Timber switch tie sets for turnouts generally conform to AREMA Plan basic number 912. However, due to the lower transit wheel loads, it is actually possible to increase the spacing of the timber switch ties in the closure rail area. As a practical matter, timber switch tie spacings should not exceed 24 inches [610 mm]. The switch and frog areas should generally remain at the AREMA spacing. When setting switch tie locations, consideration should be given to any thermite field weld locations so that welds are suspended between ties.

Timber switch ties, like timber cross ties, should be specified in accordance with current specifications of the AREMA *Manual for Railway Engineering*, Chapter 30—Ties, Part 3—Solid Sawn Timber Ties. The type of wood, tie size, anti-splitting device, wood preservative treatment, and machining should be specified in the procurement contract. As mentioned previously, designers are cautioned concerning specification of tropical hardwoods.

For additional information on timber tie special trackwork, refer to Chapter 6.

5.5.3.2 Concrete Switch Ties

Concrete switch ties are gaining wider acceptance in rail transit than was the case when the first edition of this Handbook was published. Concrete switch ties are developed to match the geometry of specific turnouts and spacing of the ties within those turnouts. Up through about the year 2000, most concrete switch tie designs in North America were designed to meet the needs of freight and passenger railroad companies and came about through the joint effort of the railroads and the concrete tie manufacturers through various technical committees. These freight and passenger/commuter railroad turnout designs were primarily for the larger turnouts (No. 15 and above) used for higher main line speeds. Relatively few switch tie design details originated on transit projects. That situation is changing as more transit agencies opt to use concrete switch ties, and designs are now available for several sizes of turnouts, crossovers, and double crossovers—including turnouts as sharp as a No. 5. If concrete switch ties are being considered for a project, particularly a small project that needs only a few turnouts of each size and hand, careful consideration of designs that have already been engineered and fabricated can save money.

Concrete switch ties for light rail transit use should be approximately 10 inches [255 millimeters] wide at the top of the tie, 11 ¼ inches [285 millimeters] wide at the base of the tie, and 9 ½ inches [240 millimeters] high throughout.

The standard embedded shoulder and elastic clip used on ordinary concrete cross ties may be used at some locations on switch ties where clearances allow the four rails to be mounted individually. Placement tolerances for the embedded shoulders are critical. The height differentials between switch, frog, and guard rail plates and the standard conventional rail installation must be considered in the cross tie design. Generally, the single rail locations have the rail seat area formed higher than the remainder of the tie so as to match the base of rail elevations in the plated areas. Threaded anchor inserts in the tie are often used to secure switch plates, frog plates, and guard rail plates, although designs that employ embedded shoulders to permit plates to be anchored using threadless elastic rail clips have become available since 2000; see Figure 5.4.3. Areas of the turnout layout where single rail installation is required, such as the closure curve zone between the heel of the switch and the toe of frog, will require an alternate rail mounting method. Some projects have specified cast-in shoulders for all special trackwork components including plated areas and conventional rail installations, thereby making many of the switch ties in the layout absolutely unique.

Concrete switch tie sets for transit use will deviate from AREMA Standard Plan No. 912, due to the lower transit wheel loads and the increased spacing required to permit tamping around wider concrete switch ties. The spacing should not exceed 24 inches [61 cm] in the switch and frog areas and 30 inches [76 cm] in the closure curve area. In the switch and frog area, the tie spacing must provide working space for thermite field welds between ties. As of this writing, there are not yet any universally accepted standards for concrete switch tie sets, and AREMA has not included any recommended concrete switch tie layouts in the Portfolio of Trackwork Plans. Instead, the tie lengths and spacings for various turnout and crossover arrangements in light rail transit track are developed by the individual projects, usually as part of the fabricator's shop drawings. See Chapter 6, Figures 6.91 through 6.9.3 for illustrations of suggested layouts for concrete switch tie turnouts. Standardization would allow for more economical engineering and manufacturing and increased use of concrete switch ties.

The concrete switch tie length should be sufficient to suit the turnout geometry and provide sufficient shoulder length. The fastenings and switch, frog, guardrail, and turnout plates should be insulated to retard stray current leakage. The concrete switch ties should comply with the appropriate specifications for concrete ties as outlined in the AREMA *Manual for Railway Engineering*, Chapter 30. For additional information on special trackwork concrete switch tie designs, refer to Chapter 6 of this Handbook.

5.6 JOINING RAIL

Rail joints are the weakest component in the track structure and are generally unavoidable on any track structure. To connect the short length (sticks) of rolled rail, a rail joint is required. There are various types of rail joints grouped as follows:

1. Welded Joints
   - Electric pressure flash butt weld

- Thermite (kit) weld

Other types of welded joints (e.g., gas pressure welding and cast welds) have been used in the past, but have been superseded by the technologies noted above. See Article 5.6.1 of this chapter and Chapter 13 for additional discussion of rail welding processes.

2. Insulated Joints

   - Non-glued bolted insulated joint—the poly-encapsulated design has superseded virtually all previous designs of non-glued insulated joints. The poly-encapsulated design is available from several vendors, and there is no reason to consider obsolete designs.
   - Glued ("bonded") and bolted insulated joint—glued joints are recommended for use in continuously welded rail.

3. Non-Insulated Bolted Joints

   - Standard bolted joint (Non-glued)—these are the ordinary bolted joints such as those that appear in the AREMA *Manual for Railway Engineering*.
   - Glued bolted joint—glued bolted joints are uncommon, but are sometimes used in locations where a permanent and robust connection is desired between two rails but it is not practical to install a thermite weld. This design requires a special joint bar that matches the contours of the full height of the rail web.
   - Pin-bolted joints—some railways have used high-strength pin bolts (also known as "Huck bolts") to fasten ordinary bolted joints together, typically with a zero joint gap so as to produce "continuous bolted rail." Keeping these joints tight can be problematic; there is no way to further tighten them because the fishing surfaces of the joint bars and rail webs wear into each other. The resulting loose joint can flex and, because the rail ends are tightly butted to each other, rail end chipping can result. This detail is not recommended unless it is also glued, as described above.

5.6.1 Welded Joints

Welded rail joints, forming continuous welded rail (strings) out of many short lengths (sticks) of the rail, have been standard for main tracks in the railroad industry for decades. Elimination of bolted rail joints has improved the track structure and reduced the excessive maintenance required at bolted rail joints. CWR strings with no bolted joints provide three specific improvements:

- Elimination of bolted joint connections and their associated mechanical problems and high maintenance costs.
- Elimination of the electrical bonding cables that are needed at bolted joints for signaling circuits and traction power return circuits.
- Reduction of the resistivity of the rail to the lowest possible level so that traction power return current is carried with the least probability of stray currents.

Rail welding in North America is generally accomplished using either the electric pressure flash butt weld or the thermite weld method.

5.6.1.1 Electric Pressure Flash Butt Weld

Most rail strings are welded together by the electric pressure welding process (commonly called "flash butt welding"). Electric flash butt welding is a forged weld created by placing an electrical charge between the slightly separated ends of the rails until the steel is plastic. The rails are then forced together to the point at which the steel refuses further plastic deformation.

There are three types of flash butt welding plants, although the basic welding equipment is the same in each case:

- Fixed welding plants are permanent installations where the rail is usually brought in and shipped out on railroad cars. Such facilities are sometimes located at or near the rolling mill where the steel rails are manufactured.
- Portable welding plants are modular units that can be brought to a project site. They typically contain most of the same equipment as the permanent plant.
- Portable welders are units typically contained within a hy-rail-equipped truck. These are usually used for making welds in track on a spot basis such as when making final closures between CWR strings; however, some contractors are now using them to fabricate strings at small project sites that can't accommodate a portable plant.

On most rail transit projects portable welding plants are used to produce strings of continuously welded rail (CWR).

Because flash butt welding forges two rail ends together under pressure, the resulting welded rail is shorter than the sum of the two individual rails from which it is made. The amount of length lost will vary by many factors, but is generally about 1 to 1 ¼ inches [25 to 32 mm] per weld. This is one reason why flash butt welding is generally impractical within special trackwork units.

The individual rolled stick rails are welded in various predetermined lengths of CWR. When CWR strings are welded for railroad service, the nominal lengths are normally 1,440 feet [439 meters], a length that has proven convenient for transport in railroad use. In rail transit work, the actual lengths of the CWR strings are usually dictated by the project's construction work plan, giving due consideration to the following:

- The space constraints of the welding location and any intermediate rail string storage locations.
- The method of transporting the CWR from the welding location (or to and from an intermediate storage location) to the installation location. CWR trains such as those used by freight railroads are uncommon in rail transit construction. In many cases of LRT construction, particularly when storage space is at a premium, the welding plant will be moved multiple times during a project, and CWR strings may only need to be moved relatively short distances using portable rollers or mini-CWR trains.
- The characteristics of the installation location. In urban areas, maintenance of highway traffic requirements will often limit work areas to the length of a city block. Hence, the

CWR strings might be both very short and fabricated in a variety of lengths to suit the field conditions.

During the electric flash butt welding process, heat-affected zones (HAZ) develop in the original rail steel on both sides of the weld immediately adjacent to the upset weld metal created during the forging. The surface hardness in the HAZ is somewhat less than either the original rail steel or the center of the weld. Under traffic, the HAZ can become dipped and battered, a condition often referred to as "mushrooming." This condition is particularly common under freight railroad loadings, but similar issues can develop in transit track, albeit at a slower growth rate. Immediate air cooling of the HAZ can increase the rail hardness and reduce the length of the HAZ; however, air cooling of the HAZ is not a standard practice as of this writing. As with many things, the track designer should not assume that rail welding contractors will automatically take certain actions. If an action is not specified in the contract, there is a significant chance it will not occur.

5.6.1.2 Exothermic ("Thermite") Rail Welding

Thermite welds are produced with molten steel cast from a crucible and poured into a specified gap between two rails. The molten steel is produced by an "exothermic" chemical reaction between aluminum and iron oxides. Additives in the mix create the other components needed to make the steel. Thermite welding requires preheating the rail ends in order to create a good bond between the rail steel and new steel produced in the thermite crucible. It is desirable that the resultant steel weld material have approximately the same hardness as the parent rail steel. Manufacturers can produce welds with different hardnesses to provide compatibility with different grades of rail steel; however, this is difficult to achieve in practice.

Similar to the electric pressure flash butt weld process, the thermite weld process also generates HAZ in the original rail steel. At least one thermite weld kit manufacturer has developed a post-weld heat treatment process that reduces this problem, but it has seen little acceptance in North America. Battered thermite welds (typically in the HAZ adjacent to the weld) are a significant problem in rigid trackforms—particularly in embedded track, which can begin to sound like jointed rail in a distressingly short time after construction. Attempts to repair this condition through surface welding are generally unsuccessful, in part because the heat of welding can damage or destroy the rail insulation system (e.g., rail boot).

One industry observer believes that many of the problems with battered thermite welds stem from the "slow bend test" mandated by AREMA. The slow bend test mandates that the weld straddling two rigid supports must be able to deflect over 1 inch [25 mm] without rupture. The apparent intent is to mimic a condition in ballasted track where the cross tie or ties beneath a rail weld have completely failed. So as to meet the slow bend test, the welding kit manufacturers need to make the weld very ductile, and much of this ductility apparently occurs in the HAZ and not the weld metal. The aforementioned industry observer notes that rail deflections on the order of those mandated by the slow bend test are virtually impossible in embedded track and that better welds might be possible if there was a modified slow bend test procedure specifically for embedded rails. European Norm EN-14370-1 might be a model for such a testing regimen. As of 2010, this issue was under consideration by AREMA Committees 4 and 12; readers should consult the current edition of the AREMA *Manual for Railway Engineering* and recent industry publications for the current status of this topic.

Prefabricated CWR rail strings are generally joined or welded together by the thermite weld process; however, many transit agencies, whenever practical, require use of a portable electric flash butt welder to join rail strings in order to eliminate thermite welds.

For additional information on electric flash butt and thermite welding during construction refer to Chapter 13.

Welding rail eliminates bolted joints and most of the associated joint maintenance. However, CWR creates other issues, such as thermal structural interaction on bridges, which must be addressed by the designer (refer to Chapter 7).

5.6.2 Insulated and Non-Insulated Bolted Rail Joints

Bolted rail joints consist of two joint bars—one on each side of the rails—that splice the abutting rail ends. They are fastened with a series of track bolts (usually six). Both non-glued and epoxy-glued rail joints are required for various conditions.

While all contemporary standards call for bolted joints to have six bolts, four-bolt joints used to be common. While four-bolt joints are usually strong enough for light rail vehicle loadings, they are not recommended because the loss of only one bolt in a four-hole joint means almost certain failure of a second bolt, particularly in CWR. For this reason, having only one functional bolt in one end of a joint constitutes a Class 1 defect under 49 CFR 213—the FRA Track Safety Standards. While 49 CFR 213 is generally not applicable to rail transit, it appears that as of this writing (2010) similar federal requirements may apply to rail transit in the near future.

At one time, various railroads had different rail drilling spacing for bolt holes; however, over the years, rail drilling spacing was standardized, as documented in the AREMA *Manual for Railway Engineering*. The hole spacing recommended in AREMA should be followed for jointed tee rails of North American design. European standards for rail drilling spacings can be appreciably different. Regardless of rail section, non-standard drilling patterns are discouraged since they complicate maintenance.

Although bolted rail joints are the weakest points in the track structure, some bolted joints are required. Most bolted joints in modern LRT main tracks will be insulated rail joints. These provide and define the signal circuit limits/sections, detect vehicle locations, define clearance points, and provide related functions. An insulated joint separates the ends of the rails to break the signal continuity by use of an insulated end post and insulation between the rails and both the joint bars and the bolt assemblies. Insulated joints are also used to isolate traction power return current paths as part of the effort to control stray current.

5.6.2.1 Non-Glued Insulated Rail Joints

These are suitable for bolted rail track and within special trackwork layouts. Bolted insulated joints (non-glued) consist of two polyethylene-coated joint bars, bolt thimbles, and an insulated end post all held together by bolt assemblies. Because these joint assemblies cannot routinely accept high tensile forces in continuously welded rail, they are recommended for use only in bolted jointed rail track.

These are the only insulated joints that are practical for field assembly under adverse weather conditions and in locations, such as crossing diamonds, where correct assembly of a glued insulated joint with the pot life of the adhesive would be problematic at best.

5.6.2.2 Glued Bolted Insulated Rail Joints

Glued insulated joints, sometimes known as "bonded" insulated joints, are similar to non-glued joints, except that the joint bars are shaped to fit the rail fishing to allow the bars to be glued to the web of the rail. The adhesive in the glued insulated joints provides a continuous shear path for CWR rail stresses to be carried through the joint so that those forces are not carried solely by the track bolts and their insulating thimbles.

Proper assembly of glued joints requires skill, good weather conditions, and the ability to keep the rail ends precisely aligned and absolutely stationary during the assembly and curing process. In the absence of these conditions, either electrical or mechanical failure of the insulated joint is very likely. For these reasons, it is recommended that glued joints be assembled in a controlled shop environment whenever possible. The resulting plug rail is then thermite welded into the proper location within the CWR after the rail has been adjusted for zero thermal stress.

For additional information on assembly of insulated joints refer to Chapter 13.

5.6.2.3 Bolted Rail Joints

Bolted rail joints can be of two designs, non-glued or glued (bonded). In light rail transit systems, non-glued bolted rail joints are typically used only in very sharp curves, maintenance yard facilities, or secondary non-revenue track.

Bolted rail has often been used on sharp curves on the premise that doing so will make it easier to replace the rail when it wears out. Arguably, the presence of the bolted joints could cause the rail to wear out faster. It's also usually not easy to replace a single rail in a sharp curve since gauge face wear on the existing rail could create unacceptable gauge line mismatch with the new rail. The theory that the internal rail stresses would be more easily controlled with bolted joints doesn't necessarily apply in extremely sharp curves since the joints are more likely to lock up, particularly if the joint bars were not precurved to match the radius. The result is often the same. Very sharply curved track often "breathes," that is, all or a portion of the track cyclically shifts in and out due to thermal expansion/contraction of the rail. This condition will leave gaps in the ballast section at the ends of the ties. Provided it happens uniformly along the length of the track curve, this is often not a concern, but it can be a major problem if the compressive rail stresses are concentrated in one area, resulting in a "sun kink."

Non-insulated glued rail joints can be used in CWR territory to maximize longitudinal load transfer through the joint bar assembly. Non-insulated glued rail joints are occasionally used in locations where a field-welded joint might be desired, but is not possible. As in the case of bonded insulated joints, a high degree of quality control is necessary during assembly.

Non-insulated glued rail joints generally have a zero joint gap with abutting rail ends. Zero gap connections have a tendency to result in chipped rail ends due to rail steel migration across the butted joint. Although bonded, each rail end tends to independently pump vertically, resulting in chipped ends. This condition can require more maintenance than thermite weld mushrooming.

When non-insulated bolted joints of any design are used in electrified rail transit tracks, it is necessary to bridge them with an electrical cable so that signal and traction power currents can pass through. For additional information on electrical bonding for signal and traction power, refer to Chapters 10 and 11.

5.6.3 Compromise Joints and Compromise Rails

Compromise joint bars are the customary method for joining two dissimilar rail sections. The compromise joint bars are machined or forged to the shape necessary to join the two dissimilar rails. The shape allows both rails to align at the top of rail and the gauge face of both rails.

Compromise joint bars, due to design shape, are virtually always made for right- and left-hand installations. The hand designation is defined by the location of the larger rail as seen from the center of the track.

To avoid abrupt changes in rail stiffness, compromise joints should not be installed between rails of greatly different heights. Generally, the difference in rail height should not exceed about 1 inch [25 mm]. If the difference exceeds that value, a transition rail of intermediate height should be used with two compromise joints. Exceptions can be made if the rails are continuously supported, as occurs in embedded track, or if rail operations occur at a very slow speed, such as within a maintenance shop.

Like any bolted rail joint in CWR track, compromise joints are subject to high tensile stresses and possible bolt failure due to high shear stress. The problem is exacerbated by the moment loading of the joint caused by eccentricity of the loading. To overcome these issues, welding of the two dissimilar rail sections can be considered. Special thermite weld kits are available for this situation, and some vendors of flash butt welding services can pressure weld dissimilar rail sections that are not too different in overall cross section.

Other options include forged steel compromise rails and compromise rails machined from a block of rail steel. The latter are often used for connections between groove rails and tee rails. **Figure 5.6.1** shows a freshly machined central block for a compromise rail between CEN 67R1 groove rail and 115 RE tee rail. The compromise rail includes a central block made of rail steel with segments of each rail section machined at opposite ends. A common top of rail and gauge line are developed during the design and machining process. The central compromise block is then electrically flash butt welded to the two short sections of the matching rails. This compromise rail block with extended rails is then welded into the track, providing a boltless connection.

5.7 BALLAST AND SUBBALLAST

Ballast, the material used to support the cross ties and rail, is an important component in the track structure. It is the integral part of the track structure in the roadbed, and the quality of the ballast material has a direct relationship to the track support system.[7]

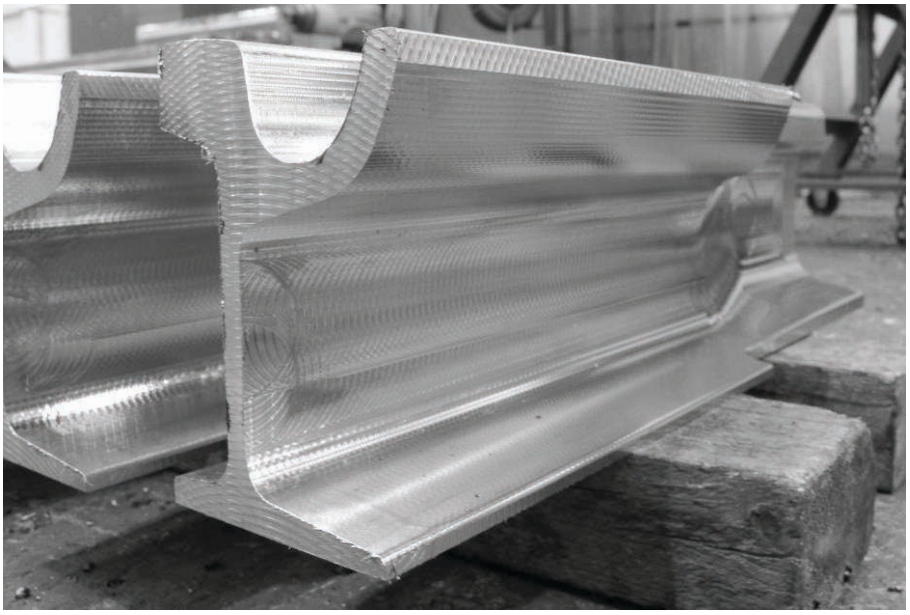


Photo courtesy of Voest Alpine

Figure 5.6.1 Machined central block for compromise rail

Light rail transit vehicles often exceed 100,000 pounds [45,500 kilograms], placing increased importance on the track structure, particularly the ballast quality and quantity. Superior ballast materials improve the track structure performance and are an economical method of increasing the track strength and the track's modulus of elasticity.

The importance of the quality and type of ballast material, along with standard test methods for evaluating the ballast material, cannot be overstated.

The quality of the ballast will be determined by the choice of rock and the eventual testing of the rock, followed by observing the performance of the track structure. The physical and chemical properties of the ballast rock or stone can be determined by many material tests and performance evaluations. However, the true test of ballast performance is to observe it in the real-life track structure.

5.7.1 Ballast

Ballast should be a hard, dense, mineral aggregate with a specific configuration of many fractured faces, an angular structure with sharp edges, and a minimum of flat and elongated particles.

5.7.1.1 Ballast Materials

As a guideline, ballast material for light rail transit use shall be as follows:

- With Concrete Cross Ties
  - Granite: a plutonic rock with an even texture consisting of feldspar and quartz.
  - Traprock: a dark-colored, fine-grain, non-granitic, hypabyssal or extrusive rock.

- With Timber Cross Ties
  - Granite and traprock, as noted above for concrete cross ties.
  - Quartzite: granoblastic metamorphic rock consisting of quartz and formed by recrystallization of sandstone or chert by metamorphism.
  - Carbonate: sedimentary rock consisting of carbonate materials such as limestone and dolomite. Carbonate ballast must never be used with concrete cross ties.
  - Blast furnace slag (commonly used for ballasting secondary tracks on freight railroads). It should never be used on electrified transit lines since the residual metals in the slag can exacerbate any problems with stray currents.

5.7.1.2 Ballast Gradation

It is important to match ballast size or gradation to the type of cross tie that will be used. The gradation of the ballast determines the sieve size to be used in the process of ballast grading. **Table 5.7.1** lists the recommended ballast gradations for light rail transit use with concrete and timber cross ties.

No. 5 ballast has been used for yard applications with timber cross ties to provide an easier walking surface. The smaller gradation may lead to earlier fouling of the ballast and eventual lack of drainage. No. 5 ballast is only recommended when the yard area is honeycombed with an underlying drainage system and substantial surface drainage channels. As an alternative, a thin layer of No. 5 ballast placed over the larger principal ballast stone can provide a safe walking surface. However, for safety and convenience, any areas where a significant number of employees can be expected to be walking, such as where train operators pick up and drop off trains, should be provided with a paved walkway surface. Paved walkways also facilitate snow removal in cold weather climates.

Table 5.7.1 Ballast gradations

Ballast Size No.	Square Opening →	3" [76]	2½" [64]	2" [51]	1½" [38]	1" [25]	¾" [19]	½" [13]	3/8" [10]	#4 [6]
	Nominal Size↓	Percent Passing								
Concrete Cross Ties										
24	2½" – ¾" (64-19)	100	90-100	–	25-60	–	0-10	0-5	–	–
3	2" – 1" (51-25)	–	100	95-100	35-70	0-15	–	0-5	–	–
Timber Cross Ties										
4A	2" – ¾" (51-19)	–	100	90-100	60-90	10-35	0-10	–	0-3	–
4	1½" – ¾" (39-19)	–	–	100	90-100	20-55	0-15	–	0-5	–
5	1' – 3/8' (25-10)	–	–	–	100	90-100	40-75	15-35	0-15	0-5

5.7.1.3 Testing Ballast Materials

Ballast material should be tested for quality through a series of tests undertaken by a certified testing laboratory. The tests should include the following:

- ASTM C88: *Soundness of Aggregates by Use of Sodium Sulfate ($NaSO_4$).* The sodium sulfate soundness test is conducted with the test sample saturated with a solution of sodium sulfate. This test will appraise the soundness of the aggregate. Materials that do not meet applicable test limits can be expected to deteriorate rapidly from weathering and freezing and thawing.

- ASTM C117: *Test Method for Material Finer than 75 microinch (No. 200 Sieve) in Aggregates by Washing (including Dust and Fracture).* The concentration of fine material below the 200 sieve in the ballast material is determined by this ASTM test. Excessive fines are produced in some types of crushing and processing operations and could restrict drainage and foul the ballast section.

- ASTM C127: *Specific Gravity and Absorption.* Specific gravity and absorption are measured by this test method. Specific gravity in the Imperial (English) measurement system relates to weight and in the metric system to density. A higher specific gravity indicates a heavier material. A stable ballast material should possess the density properties shown in **Table 5.7.2** to have suitable weight and mass to provide support and alignment to the track structure. Absorption measures the ability of the material to absorb water. Excessive absorption can result in rapid deterioration during wetting and drying and freezing and thawing cycles.

- ASTM C142: *Test Method for Clay Lumps and Friable Particles in Aggregates.* The test for friable materials identifies materials that are soft and poorly bonded and thereby lead to separate particles being detached from the mass. The test can identify materials that will deteriorate rapidly. Clay in the ballast material is determined by the same test method. Excessive clay can restrict drainage and will promote the growth of vegetation in the ballast section.

- ASTM C535: *Test Method for Resistance to Degradation of Large-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine*. The Los Angeles abrasion test is a factor in determining the wear characteristics of ballast material. The larger ballast gradations should be tested in accordance with ASTM C535, while ASTM C131 is the wear test for smaller gradations. Excessive abrasion of an aggregate will result in reduction of particle size, fouling, decreased drainage, and loss of supporting strength of the ballast section. The Los Angeles abrasion test can, however, produce laboratory test results that are not indicative of the field performance of ballast materials.

- ASTM D4791: *Test Method for Flat and Elongated Particles.* The test for flat and elongated particles uses one of three dimension ratios. Track stability is enhanced by eliminating flat or elongated particles that exceed 5% of ballast weight. Flat or elongated particles are defined as particles that have a width-to-thickness or length-to-width ratio greater than 3.

Table 5.7.2 lists the recommended limiting values for the ballast material tests. The ballast guidelines for timber and concrete cross tie applications are based on experiences with concrete cross tie ballasted track. The concrete cross tie load characteristics are quite different from the timber cross tie loadings on ballasted track. The concrete cross tie is heavier and less flexible in absorbing impact loads, thus transmitting a greater load to the ballast, which results in a higher crushing degradation load on the ballast particles. The selection of material for ballasted concrete cross tie track must be limited to granites and traprock. The selection of ballast materials for timber cross tie track can include all the materials listed in Table 5.7.2.

Table 5.7.2 Limiting values of testing for ballast material

		Ballast Material				
	Cross Tie Material	Concrete Ties		Timber Ties		
ASTM Test	Property	Granite	Traprock	Quartzite	Limestone	Dolomitic Limestone
C117	Percent Material Passing No. 200 Sieve (maximum)	1.0%	1.0%	1.0%	1.0%	1.0%
C127	Bulk Specific Gravity (minimum)	2.60	2.60	2.60	2.60	2.65
C127	Absorption Percent (maximum)	1.0%	1.0%	1.0%	2.0%	2.0%
C142	Clay Lumps and Friable Particles (maximum)	0.5%	0.5%	0.5%	0.5%	0.5%
C535	Degradation (maximum)	35%	25%	30%	30%	30%
C88	Soundness (Sodium Sulfate) 5 Cycles (maximum)	5.0%	5.0%	5.0%	5.0%	5.0%
D4791	Flat and/or Elongated Particles (maximum)	5.0%	5.0%	5.0%	5.0%	5.0%

Other test procedures exist for testing potential ballast materials, such as the Petrographic Analysis and the Ballast Box Test performed at the University of Massachusetts. The services of a qualified certified specialist and testing laboratory in the field of geological materials is recommended to further refine the material selection process and verify the suitability of stone from a particular quarry for potentially supplying ballast. The quality of stone within any particular quarry can vary depending on a number of factors, including the working face of the quarry and the amount of original overburden at that part of the site. In particular, stone excavated from higher elevations on a quarry face should be monitored closely since visual methods may be insufficient to accurately define the line between acceptable and inferior material.

5.7.2 Subballast Materials

Subballast is a granular base material placed and compacted over the top of the entire embankment or roadbed to prevent penetration of the ballast into the subgrade. Common subballast materials include crushed stone, natural or crushed gravel and sands, or a mixture of these materials. The subballast layer must be of sufficient depth and shear strength to support and transfer the load from the ballast to the subgrade.

Saturated subgrade soils are the principal reason that ballasted track surface can deteriorate. For this reason, unlike a highway subbase material, track subballast is generally graded to be somewhat impervious and shed water from its surface. Impervious subballast material should divert most of the runoff within the track area to the side ditches to prevent saturation of the subgrade.

Use of impervious subballast material sometimes requires the placement of a layer of sand between the subballast and the subgrade to release the capillary water or seepage of water below the subballast. A layer of non-woven geotextile will accomplish this as well. A qualified geotechnical engineer should make this determination based on analysis of the soils that will make up the subgrade.

5.8 HIGHWAY/RAILWAY AT-GRADE CROSSINGS

Designs for at-grade crossings of railway track by roadways vary depending on the specific type of track structure involved—ballasted, direct fixation, or embedded track—and the issues associated with their implementation. The design of any crossing must address subbase preparation, drainage, limits of crossing installation, type of crossing materials, flangeway widths, and walkway/street delineation markings. In some cases, transition sections may be appropriate between the relatively stiff track at the crossing and softer adjoining trackforms.

Providing a solid subgrade beneath railway/roadway crossings is extremely important due to the double loadings and impact of railway and roadway traffic. A geotechnical engineer should be involved in the design of the subbase with consideration of whether the natural soil materials at the site might require reinforcement or replacement so as to provide a solid base for the crossing.

Drainage is as important to road crossings as the subbase. It is imperative that surface runoff be controlled and any penetration in the crossing area be removed before the accumulation of water becomes detrimental to the subbase. Drainage of the area must be carried to the closest storm drainage catch basin or system. Drainage in embedded track must include a transverse track drain on the track profile high side of the road crossing to intersect track surface runoff and flangeway flow.

The limits of the crossing materials and adjoining roadway reconstruction must be considered along the track and extending into the intersecting street. The crossing limit along the track should include any existing or required adjacent crosswalks. The track profile grade line usually must be set to conform to the existing roadway conditions and still provide the proper geometrics for transit. The intersecting street profile, contours, and curbline elevations often must be

modified to adapt to the track crosslevel and to intercept and channel roadway surface drainage away from the track.

It generally is not recommended to construct crossings using asphalt except in instances where the roadway traffic is extremely light. Even then, modular flangeway rubber, available from several manufacturers, should be used. A better design for ballasted track utilizes modular crossing panels that span from rail to rail and also extend a little more than 2 feet [60 cm] outside of the rails. Commercially available crossing products include both precast concrete panels and rubber panels. Proper support of the panels on the outside ("field side") of the rails generally requires extra length cross ties, with 10-foot [300-cm] ties being common. Regardless of the grade crossing product used, the rail should be wrapped in rail boot so as to electrically isolate the rail from the crossing panels and the earth. LRT crossings constructed without rail boot invariably incur extreme electrolytic corrosion of the rails and rail fastenings within only a few years.

Occasionally, it might be necessary to provide a highway crossing of a direct fixation track. It generally isn't possible to use off-the-shelf modular panels designed for ballasted track as they will not provide clearance for the direct fixation rail fasteners. In such cases, a segment of embedded track may be a more appropriate choice.

The state of the art in grade crossing surface materials is constantly evolving, and the reader should contact vendors for current product information.

Modular grade crossing systems are not watertight, and it is possible for storm water runoff to convey dirt that will penetrate the crossing and compromise the methods of electrically isolating the rail. Rail boot flexure over the edges of the cross ties may eventually lead to tears and resulting electrical leaks. The crossings and their insulation systems may therefore require excessive amounts of maintenance to remain effective. For this reason, some projects have used a segment of embedded track in lieu of other types of crossings. Embedded track provides the required total rail isolation required for stray current control through the crossing area.

The flangeway width is important and current ADAAG standards for railroad and flangeway for transit use must be determined during the design. As of 2010, ADAAG requires flangeways for rail transit crossings to be no greater than 2 ½ inches [63.5 mm]. Flangeways for freight railroad crossings are permitted to be no more than 3 inches [76 mm], the same value that is recommended by AREMA. AREMA's standard is based on the fact that the allowable wear on railroad flanges under AAR rules for interchange freight cars effectively raises the track gauge/wheel gauge freeplay to a value at which the backs of the wheels could contact the crossing surface material in any flangeway less than the 3-inch [76-mm] dimension. Whether this dual standard will persist is unclear, and users of this Handbook are encouraged to verify the current requirements before commencing design.

Delineation of the walkways, the traveled roadway, and the restricted entry to the trackway must be designed to suit each road crossing intersection.

Design of the track and civil engineering elements of highway/railway crossings must be carefully coordinated with the designers of the LRT signal systems and crossing warning systems. See Chapter 10, Article 10.2.10, for additional information.

5.9 TRACK DERAILS

Track derails are operating protective devices designed to stop (by derailing) any unauthorized rail vehicles from entering a specific track zone. Generally the track zone is the operating segment of the main line. The protection is placed at all strategic track locations where secondary, non–main line operating side tracks such as pocket tracks, storage or maintenance tracks, and, in some instances, yard lead entry tracks connect to the main line. Derails are occasionally used to prevent vehicle or equipment movement onto portions of track where vehicles, work crews, or equipment are utilizing the designated track space.

Derails are placed at the clearance point of all railroad industrial tracks that connect to either an LRT joint use track or to a railroad main track. Derails are also used at other track locations where they would be likely to prevent or minimize injury to passengers and personnel and/or damage to equipment. Derails are located so as to derail equipment in the direction away from the main track.

Derails should be considered at track connections to the main line where

- The prevailing track grade of the connecting track is descending toward the main line. The secondary track is used for the storage of unattended (parked) vehicles.
- The secondary track is a storage track for track maintenance vehicles only.
- The connecting track is a railroad industrial siding.
- A railroad track crosses the LRT at grade.

In the situation of a railroad track crossing the LRT at grade, there would likely be appreciable controversy concerning which railway's vehicles are to be derailed. Derailing a freight train headed toward an LRT crossing that is already occupied by an LRV loaded with passengers might be just as disastrous as derailing the LRV. However, this may be a moot point; while there are dozens of locations in the United States where rail transit crosses a freight railroad at grade, the liability issues involved have caused the railroads to become increasingly concerned about not permitting any more such crossings.

Derails are available in various designs—sliding block derail, hinged block derail, and switch point derail (available in single switch point or double switch point rail designs). Derails are generally designed to derail the vehicle in a single direction either to the right or left side of the track. The sliding and hinged block derails consist of essentially two parts, the steel housing and the derailing guide block. The sliding derail is generally operated with a connecting switch stand. The hinged derail is operated manually by lifting the derailing block out of the way or off the rail head. The switch point derail is exactly as described, a complete switch point (or two points) placed in the track to derail when the switch point is open.

As a guideline, the type of derail to be used depends upon the site-specific conditions and type of protection to be provided. Occasionally block derails can fail, particularly when main line track is

exposed to the intrusion of heavily loaded cars, multiple car trains, track conditions that permit the intruding cars to gain momentum in advance of the derail, and tight curvature on the siding track. The switch point derail provides the greatest insurance that all wheels of the intruding vehicle will be derailed and deflected away from the main track. In addition, switch point derails are readily adaptable to standard power switch machines, which can provide positive indication to the train control system that the derail is or is not in the derailing position.

5.10 RAIL EXPANSION JOINTS

Continuously welded rail in long strings does not expand or contract with changes in temperature unless there is a designed opening or break in the rail. A continuous CWR installation introduces high thermal stress into the rail as the temperature approaches the extremes (both cold and hot) for the geographic area. Ordinarily, these stresses can be handled by the track structure. However, there are situations, typically on bridges, where rail expansion joints are installed so as to allow the rails to expand and contract to alleviate a build-up of thermal stresses.

5.10.1 Rail Expansion Joint Theory

When the track is carried by an aerial structure or bridge, the superstructure will need and is designed to thermally expand and contract with temperature. If the structure spans or deck are designed with one fixed end and one expansion end per span length, allowing a controlled, limited amount of span expansion and contraction, thermal structural expansion should not be a major concern for the rail installation. This type of installation is generally not a concern in tangent track as the elastic rail clips used in the direct fixation rail fasteners allow sufficient fastening slippage to permit the structure, including the rail fastener assemblies, to expand and contract beneath the fixed length of rail.

If the structure is designed with a limited number of joints and expansion/contraction is transferred to only one or two locations, with a sizable accumulation of expansion/contraction at these few points, there will need to be a detailed rail-to-structure interface analysis. For additional information on structure/ rail interaction, refer to Chapter 7.

In certain structures, particularly those that carry sharply curved track, the interaction between the CWR and the structure makes it desirable to limit rail stresses from the thermal structural forces. This can be accomplished by allowing the rail to freely move longitudinally within defined zones. A combination of low-restraint track fasteners and rail expansion joints allows this movement to take place safely. The use of low-restraint fasteners at structural expansion joints allows the structure to “breathe” without overstressing the rails. The rails must also be anchored or fixed between expansion zones with either a section of high-restraint fasteners or a specific rail anchor to control overall rail position. The anchoring will also control the transfer of acceleration and braking forces into the structure.

Experiences have shown that extremely sharp curves—radii below about 220 feet [67 meters]—will not consistently allow the rail to slip through the fastenings, resulting in forces being transferred directly into the structure. Again, this type of rail/structure interface should be reviewed by referring to Chapter 7.

5.10.2 Structural Configuration

Bridge trackwork that includes rail expansion joints still needs to have the rail anchored at some location. In those high-restraint areas, a conventional direct fixation fastener is utilized, and the structure is designed to accept the thermal stress loads generated by movement of the structure. The expansion or contraction of the rail emanates from the high-restraint zone through a zone of low-restraint rail fasteners and is bounded on the other end by a rail expansion joint.

Simple span bridge structures with an arrangement of fixed ("F") and expansion ("E") bearings following this general pattern:

E span F/F span E/E span F/F span EE span F/F span E

can usually be constructed using conventional high-restraint direct fixation fasteners and without any rail expansion joints. In special cases, it might be appropriate to include a short segment of low-restraint rail fasteners straddling the vicinity of each E/E pier.

See Chapter 7, Article 7.5, for additional discussion of rail/structure interaction. See Article 7.4.2.3 for detailed discussion of rail expansion joints for embedded track.

5.10.3 Rail Expansion Joint Track Details

Rail expansion joints are designed to allow for a specific length of thermal rail expansion and contraction to occur. One end of the expansion joint is fixed and connected to a rigid no-movement portion of rail. The other end consists of the expandable movable rail, which is allowed to slide in and out of a designed guideway.

5.10.3.1 Rail Expansion Joints for Open Trackforms

The most common rail expansion joint design for open track functionally resembles a fixed switch point nestled against a stock rail that is free to slide back and forth. Special expansion rail joints have been designed with dual direction rail expansions or double-ended expansion joints. **Figure 5.10.1** illustrates a double point rail expansion joint assembly suitable for use in either ballasted or direct fixation track.

Expansion rail joints in the track system present problems from both a track maintenance and an environmental perspective. Due to the slightly discontinuous running rail surface and the special trackwork sliding rail joint component, extra maintenance is required to maintain the rail joint and adjacent rails and to monitor the position of the loose rail end to ensure that sufficient space is available for further rail expansion. The specific design of the expansion rail joint within the discontinuous running rail surface introduces additional noise and vibration.

Similar to bolted rail joints, sliding rail expansion joints must be electrically bonded. The bond cable must have sufficient slack to accommodate the rail movement and also not present an obstacle to inspection and maintenance of the rail joint. In double rail expansion joints, the main bond cable would extend from one running rail to the other bypassing the central fixed portion of the expansion joint, and smaller bond cables would connect the central unit to the rails on each side. See Chapter 10 for additional information on rail bonding methods.

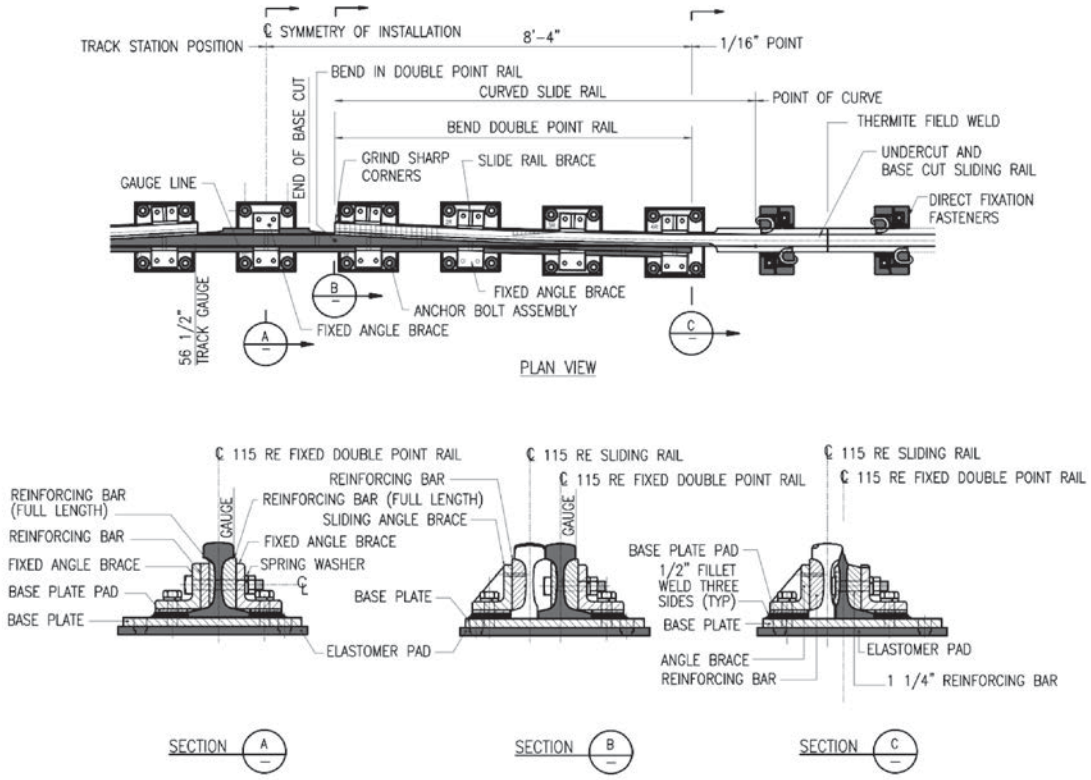


Figure 5.10.1 Double-ended sliding rail expansion joint

As a guideline, rail expansion joints in ballasted track or direct fixation track are only recommended for long bridges or aerial structures. They are also needed at the fixed span approach to a movable bridge.

5.10.3.2 Rail Expansion Joints for Embedded Track

Exceptions to the aforementioned guideline include embedded track on an aerial structure, where the rail is an integral part of the deck structure and the design does not allow the structure to move independently from the rail. In this situation, an embedded expansion rail joint at each bridge deck expansion joint of the structure is a requirement.

Embedded rail expansion joints must be carefully detailed so that they are free draining; otherwise, they can fill up with dirt and become non-functional. Since rail expansion joints are an inherently weak part of the track structure, it is recommended they be positioned entirely on one span or the other and not actually span the joint in the bridge deck.

Embedded rail expansion joints can also become a source of stray current leakage and hence endanger the underlying structure. Electrically bonding around embedded rail expansion joints without creating an unintentional path for stray currents is a challenge. On any bridge with two or more expansion joints along each rail, it is recommended that a separate negative return bond cable run around the full length of the superstructure. Doing this will help protect the

superstructure from stray current corrosion by limiting the amount of traction power return current carried by the rails on the bridge.

While embedded track can be used on an aerial structure, the complications cited above mean that it will require frequent inspection and maintenance. For this reason, the use of embedded track on an aerial structure is not recommended and should be avoided in the initial planning phase when considering the types of trackway. See Chapter 7, Article 7.4.2.3, for additional information on rail expansion joints for embedded track bridges.

5.10.4 Rail Anchorages

In conjunction with the use of sliding rail expansion joints, situations arise where it is desirable to fix the rail at a particular location so that all expansion movement occurs predictably in one direction only. Rail anchorages (not to be confused with the rail anchors used in conventional timber tie ballasted track) are devices that are securely clamped to the rail and also anchored to an underlying structure. The design of the rail anchorage should be sufficiently robust to handle the thrust associated with the length of rail to be controlled. **Figure 5.10.2** illustrates a rail anchorage for medium duty application.

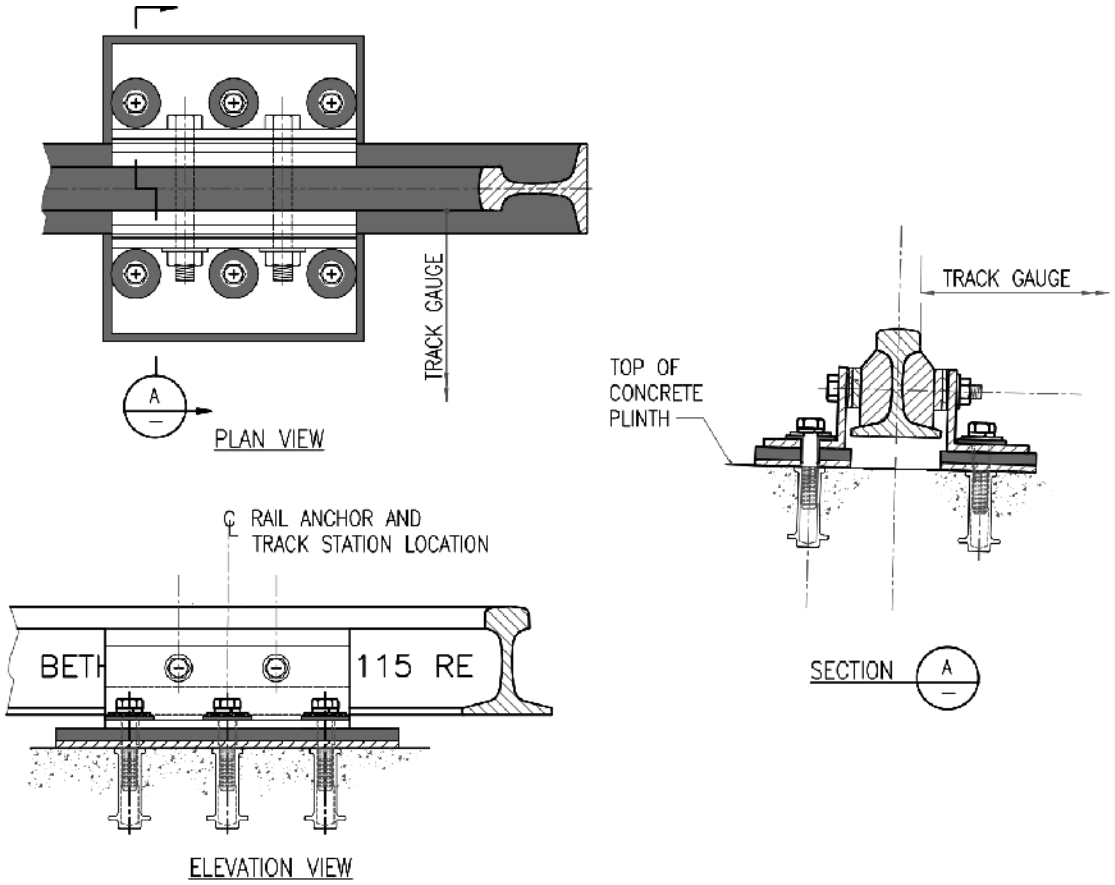


Figure 5.10.2 Rail anchorage

5.11 END-OF-TRACK BUMPERS AND BUFFERS

As important as the tangent and curved track is throughout the transit system, the end of track cannot be overlooked. Bumping posts, friction buffers, and hydraulic buffers are therefore used to prevent an out-of-control vehicle from overrunning the end of the track. The device used should be able to bring the vehicle to a stop with minimal damage and injury to the vehicle and persons on-board.

The issues for end-of-track equipment include the following:

- Warning signs and signals in advance of the end of track.
- The type of end-of-track device used. Options include the following:
  - Fixed non-energy-absorbing devices (fixed bumpers).
  - Fixed energy-absorbing devices (hydraulic buffers).
  - Friction energy-absorbing devices (Friction buffers).
- Provision of sufficient space in the civil/architectural design for the end-of-track device.
- The match between the striking face of the end-of-track device to points on the vehicle body that are designed to accept the impact forces.

Candidate end-of-track devices for various situations include the following:

- Main Line End of Track (Ballasted-Direct Fixation): friction/sliding end stop with resetting shock absorber if track sliding distance is available.
- Main Line End of Track (Embedded): Same as above, if conditions warrant, or a resetting track stop anchored to the substrata.
- Main Line End of Track (Aerial-Direct Fixation): friction/sliding end stop with resetting shock absorber. Track distance must be provided, sometimes requiring that the structure be extended.
- Yard Tracks (Maintenance Tracks): fixed non-energy-absorbing devices, such as bumping posts, anchored to the track.
- Stub-End Yard Storage Tracks and Main Line Pocket Tracks: resetting fixed devices anchored to the track.
- Maintenance Shop Tracks: Fixed resetting energy-absorbing device anchored to the structure floor (non-movable).

While fixed bumpers are commonly used in railroad service and occasionally on stub-end tracks in transit yards, they can result in significant damage to the vehicle even when equipped with a spring-mounted striking face. Therefore, the ends of track devices used at the ends of revenue service tracks should generally be of a type that gradually brings the vehicle to a stop over a appreciable distance, The two principal types of buffers in common service dissipate the kinetic energy of the moving train through either friction, hydraulics, or a combination thereof.

All end-of-track devices take up a certain amount of track length. In particular, sliding friction buffers require a significant length of track beyond the striking face. In addition, there should be a distance of not less than 15 feet (4 meters) between the striking face of any end-of-track device and the desired normal stopping location of the LRV. This length must be accounted for in the initial planning of the project, particularly if the end of track is close to the terminal station platform. The position of the end-of-track device must also be coordinated with the OCS designers, as they frequently wish to place a catenary dead-end pole immediately beyond the end of the rails.

The striking face of the bumper/buffer and how it interfaces with the vehicle is an important design feature. Unlike older LRVs, current vehicles generally do not have an exposed anticlimber that can be engaged by a matching striking head. LRVs with "bumpers" that conceal the coupler and are also designed around crash-energy management principles require different striking head configurations and quite possibly more than one head. Practitioners are encouraged to confer with vendors for the most recent information since the state of the art for bumpers and buffers is a dynamic situation.

5.11.1 Warning Signs/Signals

With ideal conditions, alert operators, no mechanical vehicle or signal failures, and a well-illuminated warning sign or signal, a train operator should be able to bring the vehicle or train to a safe, controlled stop well short of the end of track.

If the track with the end-of-track device is signaled, it will be necessary to coordinate the design of the end-of-track device with the train control system designers. In such cases, it will typically be necessary to insert at least one insulated joint in the rail ahead of the bumper. Two insulated joints will be necessary if the end-of-track device itself is grounded. Hydraulic buffers are often grounded so as to protect the sliding surfaces in the hydraulic pistons from corrosion. The insulated joints are typically positioned beyond the normal, non-emergency stopping location of the rail vehicle. Some friction buffers have insulation incorporated in the attachment points to the rails, eliminating the need for insulated joints.

5.11.2 Fixed Non-Energy-Absorbing Devices

Most fixed non-energy-absorbing end stops (bumping posts) do no more than delineate the end of track. The end stops appear sturdy because they are bolted to the rail; however, they have little ability to absorb anything but a very minimal amount of kinetic energy. Impact often results in breaking of the rail, potential derailment, and damage to the vehicle.

A positive fixed non-energy stop will halt heavy vehicles or train consists at the expense of vehicle damage and personnel injury. These stops consist of solid concrete and steel barriers generally located at the end of tracks and are generally found in freight yards and older railroad stations. Such devices are not recommended except on freight-only tracks when the railroad's standards include such units.

5.11.3 Fixed Energy-Absorbing Devices

The end stop is the point of impact, the location where kinetic energy has to be dissipated. The kinetic energy is determined considering the mass or weight of the vehicle or vehicles (train) and the velocity of the vehicle or train. Vendors of buffers have design formulas for calculating the requirements for a particular installation. For example, kinetic energy (KE) can be calculated using the following formula (these design guidelines typically use only S.I. units since the buffer manufacturers are usually overseas businesses):

$$KE = \frac{M \times V^2}{2} = \frac{200{,}000\text{kg} \times (4.47)^2}{2}$$

$$= 1{,}998{,}090\text{J or } 1{,}998\text{kJ}$$

where M = mass of the vehicle or train

= 200 Tonnes (1 Tonne = 1000 kg)

V = velocity of vehicle or train

4.47 meter/second (10 MPH)

To absorb this amount of energy without causing severe injury to the operator or passengers, an acceptable deceleration rate must be selected. The transit agency should select the rate of deceleration considering the likelihood of injury to passengers and operators and damage to the vehicles, third parties, and surrounding structures. Each agency's requirements are studied individually and are site specific. As a guideline, a deceleration rate of 0.3 g is generally considered to be acceptable.

Assuming the 0.3 g deceleration rate is selected, the next decision is to determine the type of end stop capable of providing this deceleration rate. To absorb 1,998 kJ of kinetic energy (calculated above) at a deceleration rate of 0.3 g, the distance traveled after initial impact would have to be 3.39 meters (11.12 feet), calculated in the following manner:

$$\text{Distance} = V \bullet t + \frac{d \bullet t^2}{2}$$

V= velocity of train in m/sec.

t= time to stop in seconds

d= deceleration rate ($x \bullet 9.81$ m/sec^2)

x= deceleration negative rate (selected)

$$t = \frac{V}{d^2} = \frac{4.47}{0.3 \times 9.81 \text{m/sec}^2}$$

$$= 1.52 \text{ seconds}$$

$$\text{From Above Distance} = V \bullet t + \frac{d \bullet t^2}{2}$$

$$= (4.47 \bullet 1.52) + \frac{(0.3 \times 9.81) \bullet (1.52)^2}{2}$$

$$= 3.39 \text{ meters (11.12 feet)}$$

5.11.3.1 Non-Resetting Fixed Devices

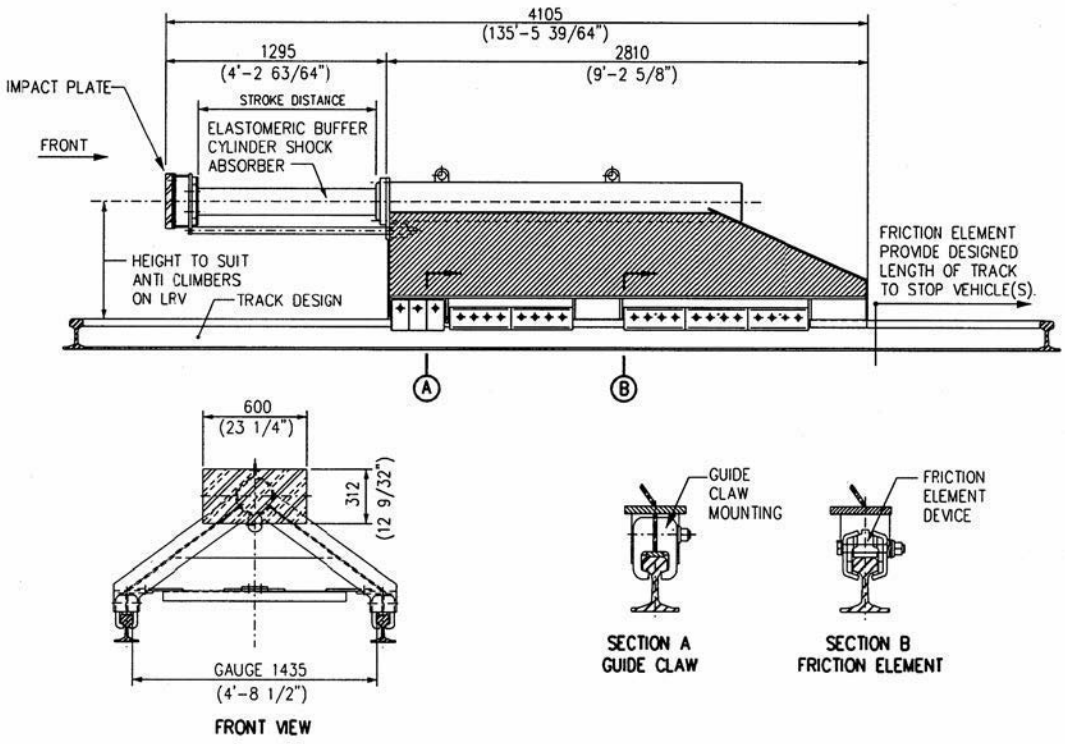
Non-resetting fixed devices (bumping posts) include sand traps, ballast mounds, and timber tie stops. These devices dissipate the kinetic energy upon vehicle impact. Sand traps and ballast mounds are effective in stopping large loads or trains; however, derailment of the initial vehicle is inevitable. Under severe cold weather conditions, the sand and ballast can freeze, reducing the cushioning effect and possibly causing additional vehicle damage. The barrier would have to be rebuilt after experiencing an impact. Any delay in reconstructing the device would leave the end-of-track unprotected for some period.

5.11.3.2 Resetting Fixed Devices

Resetting fixed devices are self-resetting and contain an energy-absorbing feature such as a hydraulic, elastomeric, or spring shock absorber. Resetting stops are limited in the amount of energy the shock absorber can dissipate and the stop structure's capability to withstand the forces at impact. As noted above, the displacement distance of the stop at impact governs the magnitude of g force—the longer the distance, the lower the g force. The anchoring stability of the end stop to the substrate governs the amount of energy that can be absorbed by the stroke of the shock absorber.

5.11.4 Friction (or Sliding) End Stops

Friction type end stops absorb the kinetic energy of stopping a vehicle or train by sliding along the end of track (see **Figure 5.11.1**). This sliding action converts the energy to friction heat at the rail surface.



(Illustration courtesy of H. J. Skelton, Ltd.)

Figure 5.11.1 Friction energy buffer stop

Friction end stops consist of two types:

- Units that are clamped to the rail.
- Units that are mounted on skids that slide with the weight of the vehicle upon them, dissipating the energy between the skids and the concrete base of track structure.

Friction end stops have the highest energy absorption capacity of all end-of-track devices. Friction stops can be designed to cover a wide range of energy absorption situations from single-vehicle to multi-vehicle trains of various mass. Impacted friction end stops move when struck and typically need to be manually reset after use; however, they are designed so that this can be accomplished with relative ease. Sometimes a friction end stop will be combined with an automatic resetting device, allowing the unit to accept light impacts without moving the friction end stop while providing the higher friction end stop protection for higher speed impacts.

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CHAPTER 6—SPECIAL TRACKWORK

6.1 INTRODUCTION

Light rail vehicles, like all steel-flange-wheeled railway equipment, need to be able to transfer from one track to another or to cross intersecting tracks. The fabricated track components and accessories needed to support and direct the rail car at these locations are collectively called special trackwork. It is presumed that most readers of this chapter are generally familiar with the layout and use of common special trackwork terms. Readers who are new to the topic can find a brief primer on basic concepts and terminology in Article 6.2.1.

The standard North American references for special trackwork are Chapter 5 of the *Manual for Railway Engineering* [1] and the *Portfolio of Trackwork Plans* [2], both published by the American Railway Engineering & Maintenance-of-Way Association (AREMA). While the *Portfolio of Trackwork Plans* currently (2010) includes some details for special trackwork on heavy rail metro transit systems, there are pronounced differences between requirements for special trackwork for light rail transit (LRT) systems and those AREMA details. In general, designers can expect to find that special trackwork design requirements on a light rail system will be more numerous and more complex than those encountered on freight railroads. In addition, there are fewer experienced vendors for transit special trackwork than for freight railroad turnouts and crossovers.

Most turnouts that are available for tangent track are standardized for simplified manufacture and installation, both for original equipment and replacing worn components. These turnouts are intended for installation in tangent track, without any vertical curvature. One of the most common track design deficiencies is the placement of turnouts within horizontal or vertical curves. Construction and maintenance of curved track is difficult and expensive. Superimposed special trackwork only exacerbates those problems. It is therefore recommended that standardized trackwork be used on horizontally and vertically tangent track whenever possible.

Light rail systems that are located in urban streets, particularly those with narrow rights-of-way, often have extremely sharp curves. This constraint often requires light rail special trackwork to be designed for a specific location, with unique parts.

6.2 DEFINITION OF SPECIAL TRACKWORK

Special trackwork is customarily defined as “all rails, track structures and fittings, other than plain unguarded track, that is neither curved nor fabricated before laying.”[1] Hence, any track can be considered special trackwork that is built in whole or part using rails that are machined, bent, or otherwise modified from their as-rolled condition. This includes any additional track components that may take the place of rails in supporting and guiding the wheels, as well as miscellaneous components that may be attached to the rails to fulfill the functions required. The term is often contracted and called simply ‘‘specialwork.’’

In general, the following items are customarily included in special trackwork:

- Turnouts and crossovers, including switches, frogs, guard rails, stock rails, and closure rails; rail fastening assemblies unique to turnouts; and miscellaneous components associated with turnouts, including switch rods and gauge plates. Crossover tracks, double crossovers including the central crossing frogs or diamond area, and single and double slip switches are included in this category. The cross ties to support turnouts and crossovers can also be considered part of special trackwork, especially concrete switch ties, which require far more design and fabrication effort than ordinary timber switch ties.
- Track crossings that permit one track to cross another at grade. Such crossings can be designed as a rigid block or can include movable center points. By definition, slip switches include a track crossing.
- Ladder track layouts where a series of turnouts are closely grouped to form a continuous entry/exit layout together with adjoining connecting closure curves into parallel tracks. Oftentimes, such layouts on transit projects will occur in areas of constrained right-of-way and will require curved frogs.
- Split switch derails (single- or double-point rail).
- Restraining rail, either bolted to a parallel running rail or supported independent of the running rail.
- Shop-curved rail of any type, including rails that are precurved in the horizontal plane, the vertical orientation, or both.
- Compromise rails for transitioning from one rail section to another, such as when a project uses both 115RE tee rail and a groove rail section.

Turnouts, crossovers, and track crossings will be addressed directly in this chapter. Information on restraining rail and shop-curved rail can be found in Chapters 4 and 5.

6.2.1 Basic Special Trackwork Components

The most common form of special trackwork is the *turnout,* which is an assembly of track components that collectively permit two tracks to merge with each other. A simplified layout of a turnout is illustrated in **Figure 6.2.1**. The turnout itself consists of several fundamental component elements as discussed below.

6.2.1.1 Switches

The switch point rails (often called either the switch points or the point rails) are the movable rails that flex back and forth and intercept the wheel flanges to direct them to the appropriate track. In its usual form, a switch point rail consists of a plain tee rail that has been pre-bent and then machined into a tapered shape that is sharp at the switch point end. This pointed end is known as the "point of switch." The opposite end is known as the "heel of switch." Switches come in various lengths and can be either straight or curved. In general, the longer the switch point rail, the more gradual the angle of divergence from the main track and the faster the rail vehicle can travel through it. The switch point rails, together with the stock rails (described below) and associated fastenings and mechanisms, are collectively called "the switch." In ordinary conversation, it is common to use the word "switch" when referring to a "turnout," which is

technically incorrect. In addition, the rest of the English-speaking world uses the word “points” for what North American track designers call a “switch.”

The stock rails are the running rails immediately alongside of the switch rails against which the switch rails lay when in the closed position. The stock rails are otherwise ordinary rails that are machined, drilled, and bent as required to suit the design of the turnout switch and the individual switch point rails.

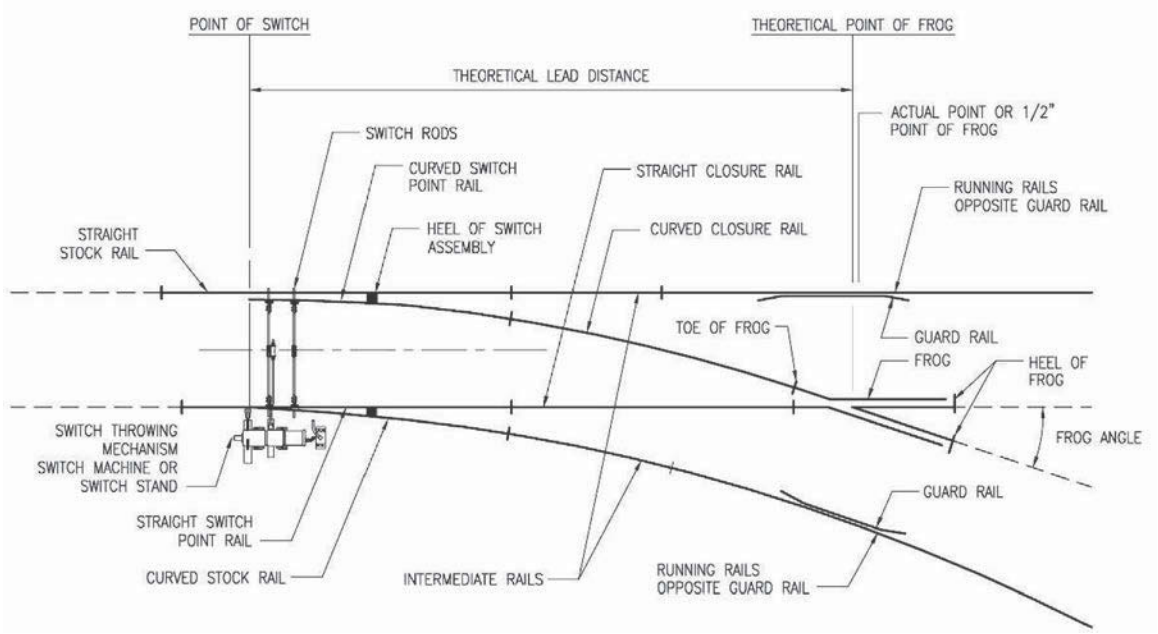


Figure 6.2.1 Turnout layout

6.2.1.2 Frogs

The *frog* is a component placed where one rail crosses another. The rest of the English-speaking world calls such units by the more obvious term “crossings.” It is believed that the term “frog” originated due to a vague resemblance between the track appliance and a feature on the sole of a horse’s hoof called the “frog,” as shown in **Figure 6.2.2**. The equestrian feature in turn was apparently named after some resemblance to the amphibian. North American trackmen in the mid-19th century, all of whom were very familiar with horses, apparently started calling the track device a frog due to the resemblance to the horse’s foot.

Openings called flangeways must be provided through the top surface of a track frog so that the flanges on the vehicle wheels can pass through. The intersection of the gauge lines of the two intersecting rails is known as the “theoretical point of frog.” The theoretical point of frog would be a sharp tip that would quickly wear and fracture in service. Therefore, the intersecting rails are cut back a short distance to a location known as the “actual point of frog,” where the metal will have enough rigidity to withstand the effects of service wear. Typically, this is a position where the point is ½ inch [13 mm] wide , leading to the common alternate terminology “half-inch point of frog.” The end of the frog closest to the switch rails is known as the “toe of frog”; the opposite end is known as the “heel of frog.”

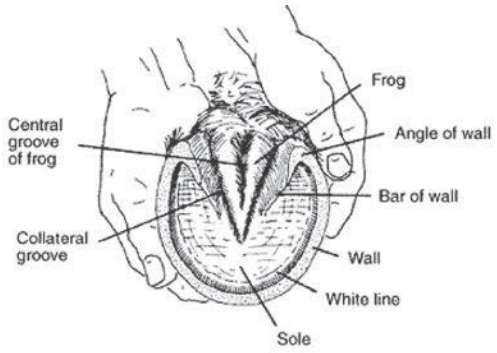


Figure 6.2.2 Frog on a horse's hoof

Typically, both rails passing through a frog are straight, although it is possible for one or both rails to be curved.

In North America, straight frogs are commonly designated by a number that indicates the ratio of divergence of both rails from a common frog centerline, as illustrated in **Figure 6.2.3**. In a No. 6 frog, the two rails will diverge at a ratio of one unit laterally for every six units of frog length. In a No. 8 frog, the divergence ratio will be one to eight, etc. The higher the frog number, the more acute the angle of divergence of both it and the turnout and the faster the rail vehicle will be able to travel through it. Mathematically, the frog number is one-half the cotangent of one-half the frog angle.

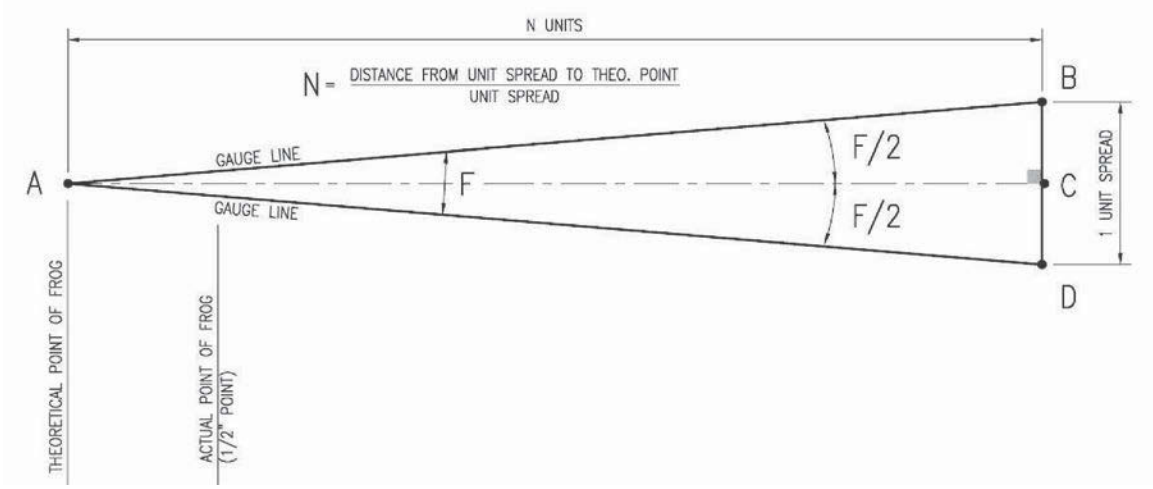


Figure 6.2.3 Frog angle—North American practice

In most of the rest of the world, the number associated with a turnout varies with the angle of divergence of the branching track from the main line track. Essentially, the number associated with a turnout is based on the tangent of the angle of the crossing while in North America it's based on the tangent of one-half of the angle of the frog. So, while in North America, a No. 6 frog has an angle of $9^{o}31'38"$, the crossing associated with a European-style 1:6 turnout has an angle of $9^{o}27'44"$ instead. In addition, much of the rest of the world often utilizes grads (a full circle has 400 grads), not degrees (a full circle has 300 degrees), for angular measurement. Therefore,

when using information from sources outside of North America, it is critical to understand the dimensional units being employed. It's not always obvious.

While straight angle (tangent) frog legs are customary in railroad work, there are often times in rail transit work when it is desirable and occasionally mandatory that one leg of the frog be curved. This happens most often in embedded track in urban areas, but having one leg of the frog be curved can also be advantageous when developing yard track ladder layouts on a constrained maintenance facility and storage site.

Due to the requirement for the wheel to pass through the open flangeway at the point of frog, the wheel tread and frog wing rail surface locations produce high impact forces, noise, and vibration. To avoid such problems, it is very important that the wheel be properly supported during its passage through the open throat area at the point of frog. The width of the wheel tread is critical in this regard. Wide flangeways combined with narrow wheel tread widths could result in loss of wheel tread contact with the top of frog, allowing the wheel to drop into the open flangeway. These design concerns are the reason for the many variations of frogs, including spring frogs, movable point frogs, flange-bearing frogs, and "lift over" or "jump" frogs. These designs are described later in this chapter.

6.2.1.3 Other Turnout Components

Other turnout components include the following:

- *Closure rails* are the straight or curved rails that are positioned in between the heel of switch and the toe of frog. The length and radius of the turnout's curved closure rails are dictated by the angles at the heel of switch and the frog. Combinations of short switches with large angles and large angle frogs will result in a sharp radius curve through the closure rail area, limiting vehicle speed. The distance between the point of switch (PS) and the actual point of the frog (PF) measured along the straight or main track closure rail is known as the turnout lead distance. (The distance to the theoretical point of frog typically appears only in calculations and is not always included on construction drawings.)

- *Guard rails* are supplemental rails, placed inboard of the main running rails, which provide supplemental guidance to the back face of the rail vehicle wheels. Guard rails form a narrow flangeway to steer and control the path of the flanged wheel. Guard rails are typically positioned opposite the frogs so as to ensure that the wheel flange does not strike the point of frog or jump to the "wrong" flangeway.
- *Heel block assemblies* are units placed at the heel of the switch that provide a splice with the contiguous closure rail and a location for the switch point rail to pivot at a fixed spread distance from the stock rail. Elaborate designs of switch heels have been introduced based on CWR installations and are described later in this chapter.
- *Switch point rail stops* act as spacers between the switch point rail and the stock rail. Stops laterally support the switch point from flexing laterally under a lateral wheel load and thereby possibly exposing the open end of switch point rail to head-on contact from the next wheel.
- *A switch operating device* moves switch rails. Switch rails can be thrown (moved) from one orientation to another by either a hand-operated (manual) switch stand or a mechanically or electro-mechanically (power-operated) switch machine. In both cases, the operating

devices are positioned at the beginning of the turnout opposite the switch-connecting rods near the point of the switch rails.

6.2.1.4 Special Trackwork Layouts

Arrangements of individual turnouts can create a variety of track layouts, thereby permitting many alternative train-operating scenarios beyond the simple divergence offered by a simple turnout:

- A *single crossover* (see **Figure 6.2.4**) consists of two turnouts positioned in two tracks that allow the vehicle to go from one track to another. The two tracks are usually, but not always, parallel, and the turnouts are usually identical.

  A pair of single crossovers—one right hand and one left hand—that are arranged sequentially along the tracks is called a *universal crossover*. This provides the maximum operational flexibility at the least cost for both trackwork and the overhead contact wire system.

- A *double crossover* (see **Figure 6.2.5**)—sometimes called a *scissors crossover*—consists of two crossovers of opposite hand orientation superimposed upon each other. In addition to the four turnouts involved, a track crossing diamond is needed between the two main tracks.

  A double crossover is typically used only when it is necessary to be able to switch from both tracks to the other in either direction, but there is insufficient space to install a universal crossover as described above. A double crossover is appreciably more expensive than a universal crossover because of the crossing diamond and the additional catenary system hardware required. Double crossovers on tight track centers can create a great deal of difficulty for the OCS designers. Maintenance expense and the downtime associated with maintenance activities are also greater for a double crossover than for a universal crossover. Nevertheless, many rail operations personnel prefer a double crossover to a universal crossover because the overall interlocking area is shorter; hence, trains will clear the interlocking area a few seconds faster. It is arguable whether, in most locations, the few seconds is worth the extra expense.

  Double crossovers between parallel tracks at a track center spacing in the range of 13 feet 6 inches to 14 feet 6 inches [4.0 to 4.3 meters] will typically have the end frogs of the diamond positioned directly opposite the turnout frogs. In some situations, the open throat area ahead of the points of frog may be completely unguarded. Should a double crossover be required, it is best to avoid track centers in the aforementioned range.

- *Track crossings, a*s the name implies, permit two tracks to cross each other. Track crossings are often called either *crossing diamonds* or simply *diamonds,* due to their plan view shape (see **Figure 6.2.6**). The intersecting angle between the two tracks can be 90 degrees or less, but rigid crossings under approximately 10 degrees are rarely encountered. In its simplest form, a track crossing is simply four frogs arranged in a square or parallelogram. The tracks through a crossing can be either straight or curved. Straight tracks are preferred since it makes the unit symmetrical, thereby simplifying design, fabrication, and maintenance. If the crossing angle between straight tracks is 90°, then the four frogs will be identical. If the angle is not 90^{o}, then the crossing will be elongated along

one diagonal axis called the “long diagonal” and the “end frogs” will be different from the “center frogs.”

If the angle of the intersecting tracks is less than that in a No. 6 frog (9^{o} 31’ 38”), it is usually necessary to use a movable point crossing. Movable point crossings incorporate movable rails in the two frogs closest to the center of the crossing. Depending on the position of these movable rails, a flangeway will be provided for one track or the other, but not both simultaneously. Movable point frogs are needed on flat-angle crossings since it is otherwise impossible to ensure that the wheel flange will follow the correct flangeway path through the center frogs of the crossing diamond. The moving point rails in a movable point crossing open and close against bent rails called *knuckle rails* and are usually operated by the same type of machines that are used to operate switches.

If it is necessary to be able to switch from one track to another at a flat-angle crossing, and space constraints make it impossible to provide separate turnouts outside of the limits of the diamond, a *slip switch* can be installed. A slip switch superimposes two switches and curved closure rails on top of an elongated track crossing, as shown in **Figure 6.2.7**. A *double slip switch* provides that same routing capability along both sides of a track crossing, as shown in phantom line on the figure. Slip switches are expensive to fabricate and install and difficult to maintain and their use should be considered only as a last resort.

Lapped turnouts can be used to achieve a more compact track layout in constrained locations. In a lapped turnout, as seen in **Figure 6.2.8**, the switch rails for a second turnout will be placed between the switch and the frog of the initial turnout. This introduces a third frog where a closure rail of the first turnout crosses a closure rail of the second turnout.

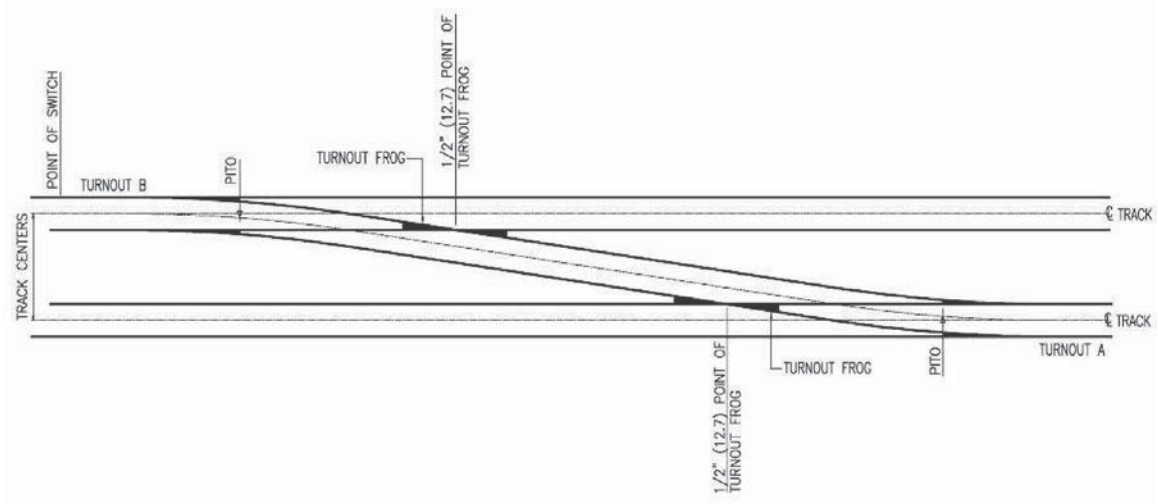


Figure 6.2.4 Single crossover (right hand)

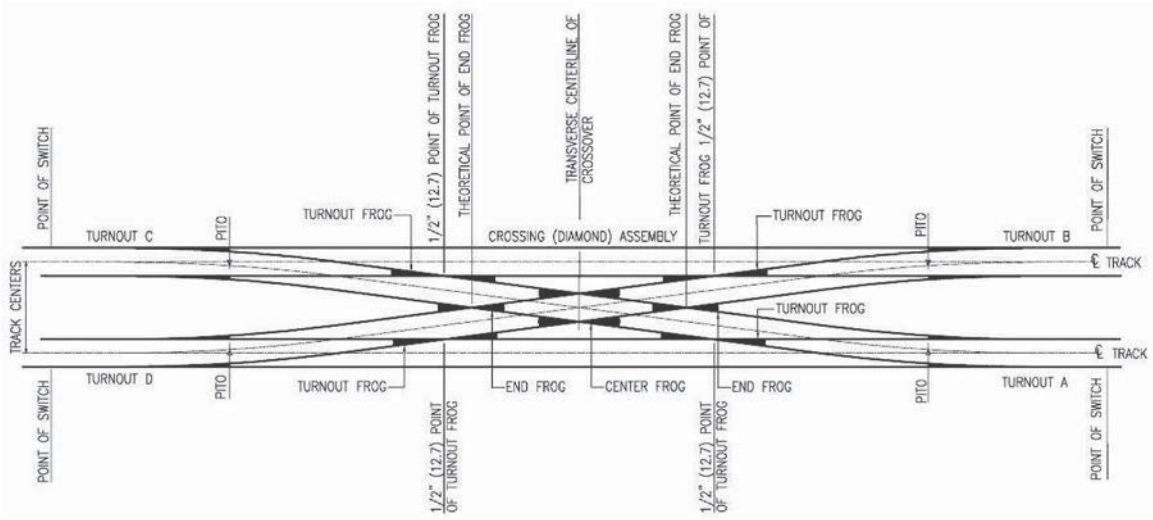


Figure 6.2.5 Double crossover

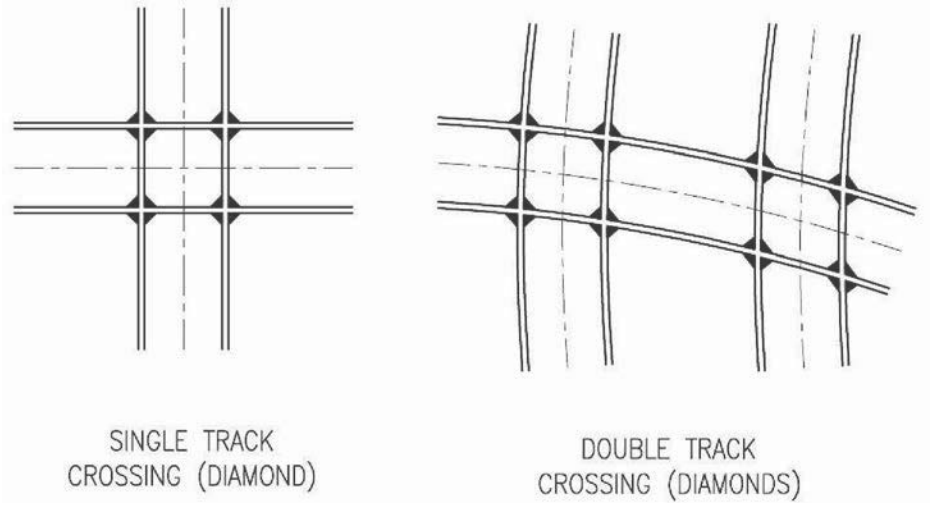


Figure 6.2.6. Single-track and double-track crossings

Combinations of turnouts and track crossings are used to produce route junctions. A common junction between two double-track routes will consist of two turnouts and a crossing diamond, the latter allowing the inbound track of one branch to cross the outbound track of the other.

The most complex junctions occur in urban areas when two double-track light rail lines intersect. **Figure 6.2.9** illustrates a full "grand union," an extremely complex arrangement that permits a rail

vehicle entering a junction from any direction to exit it on any of the other three legs. Such layouts were common on legacy streetcar systems, but are rarely seen on modern LRT. A more common LRT junction resembles a "T" intersection and would require a "double wye" (see **Figure 6.2.10**) to provide the same routing flexibility. Such layouts are often called "half grand unions," but reference to trackwork catalogs from the early 20th century[3] reveals that terminology to be incorrect as a true half grand union would actually have two additional turnouts plus four 90-degree crossing diamonds.

Complex intersections, such as grand unions, require equally complex overhead contact system wire layouts with additional poles and a spider web of pull-off wires. Beyond the expense of constructing and maintaining such complex OCS layouts is the fact that their complexity makes them visually intrusive and arguably objectionable. If a junction must occur in an area, such as a central business district, where visual aesthetics are an issue, it may be preferable or less objectionable to configure the tracks in a manner that divides the tracks and the junction turnouts over an area of several city blocks. Note this would disperse the noise and vibration generated at the special trackwork, which would otherwise be concentrated at a single intersection, and that could be perceived as either desirable or undesirable. Any such alignment alternatives should be evaluated in the early stages of the project, when the environmental impact assessments are being performed on optional route alignments, so as to be included in that analysis.

Lapped turnouts, double crossovers, movable point crossings, slip switches, and double slip switches are all very costly to design, fabricate, install, and maintain. A more economical track system is achieved when the special trackwork consists only of turnouts, single crossovers and simple track crossings.

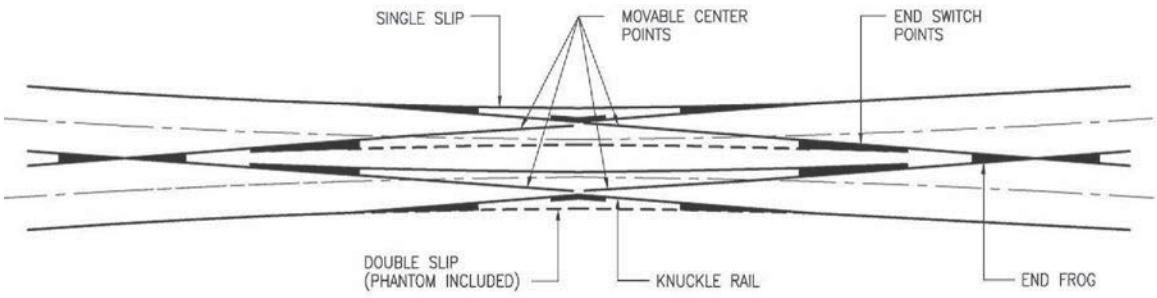


Figure 6.2.7 Single slip switch

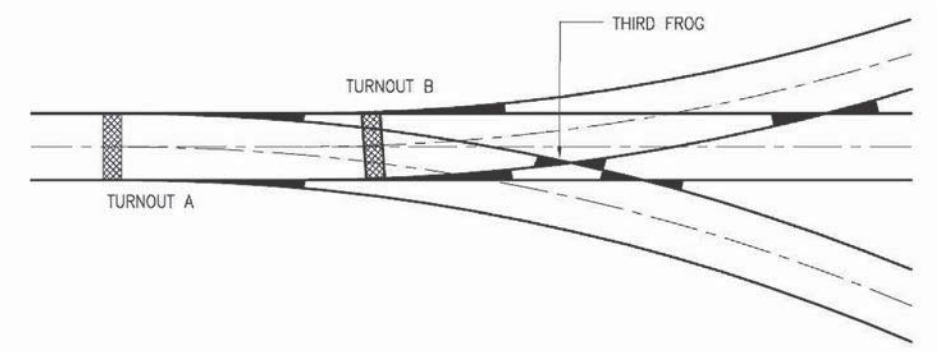


Figure 6.2.8 Double switch lapped turnout—three frogs

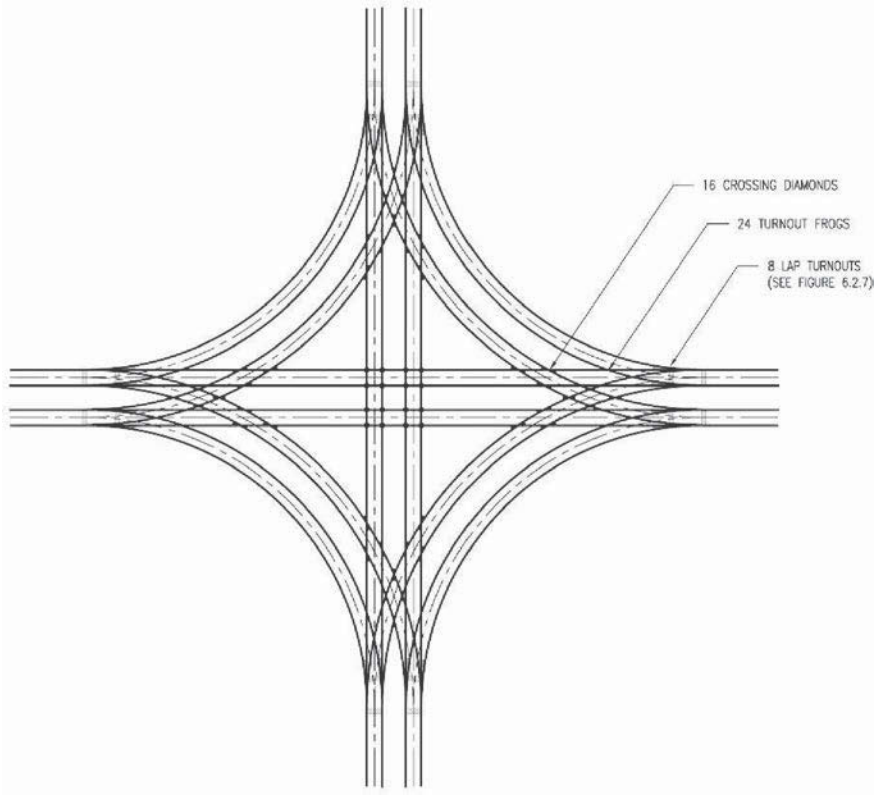


Figure 6.2.9 Full grand union

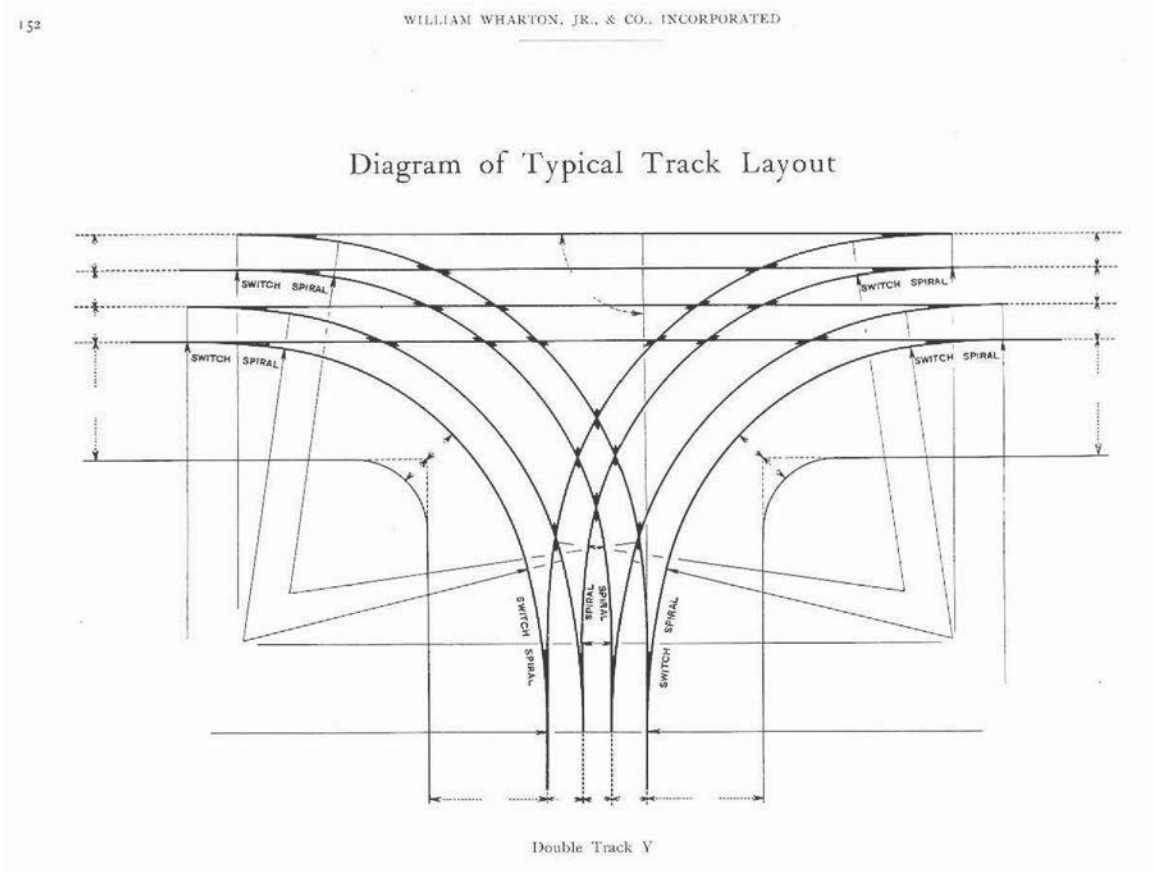


Figure 6.2.10 Double wye [3]

6.2.1.5 Non-Symmetrical Special Trackwork Layouts

Track alignment engineers, often rightfully, consider their work to be an art and are therefore fond of smooth alignments and symmetry. In the case of special trackwork, symmetry is generally preferred so as to avoid non-standard configurations that include one-of-a-kind components, thereby increasing initial procurement costs and requiring the stocking of unique spare parts. However, there are often instances in which complete symmetry is not possible. Non-symmetrical and unconventional arrangements of individual turnouts can create a variety of track layouts, thereby permitting alternative train-operating scenarios that might not otherwise be possible. Some examples include the following:

- If a crossover is required between two tracks that are not parallel, there are two options. The first option would be to configure the layout with two equal turnouts, but to insert a curve between the two turnout frogs with a central angle equal to the difference in the main track bearings. This curve should be designed with as large a radius as possible, yet not interfere with the heel end of the turnout frogs. The second option would be to use two different turnouts, such as a No. 6 and a No. 8. This can be advantageous if the angle of divergence of the main tracks is, or can be, made identical to the difference in the frog angles. However, the design speed for the crossover movement will be restricted to that of the smaller turnout. In both options, the design is greatly simplified if both tracks are on

identical, parallel tangent track grades; otherwise, a vertical curve may be required to compensate.

- Double crossovers are usually designed with symmetrical layout along the centerline of the alignment. However, there may be occasions where the layout design is not symmetrical, where it is beneficial to offset the points of switch in one track relative to the other, thereby laterally offsetting the crossing diamond location relative to the two tracks. Typically, this might be considered in situations where the track centers are narrow, with the result that there is insufficient space for the switch machines to be directly across from each other. Another example that is seen somewhat often is symmetrical configuration of the crossing diamond of a double crossover, but with one or more of the turnouts being the opposite hand of the usual arrangement, thereby creating a divergence equal to the turnout frog angle.

Judicious use of non-symmetrical layouts can sometimes resolve a seemingly intractable alignment problem and thereby make it possible to meet the requirements of the operating plan.

6.3 LOCATION OF TURNOUTS AND CROSSOVERS

The ideal design locations for turnouts, crossings, and crossovers are flat and straight sections of track. If special trackwork is installed in track with horizontal curves, superelevation, or vertical curves, the ability of the trackwork to perform in a satisfactory manner is compromised. Trackwork designers should work closely with their counterparts who are defining transit operations requirements and setting route geometry so that turnouts and crossovers are not placed in difficult locations and the overall requirements for special trackwork are minimized.

6.3.1 Horizontal Track Geometry Restrictions

As switches and frogs unavoidably create discontinuities in the running surface of the track structure, a disproportionate number of derailments occur at or near special trackwork. It is therefore extremely important to carefully consider the track geometrics approaching, passing through, and departing from special trackwork.

6.3.1.1 Track Geometry in the Vicinity of a Switch

Switch point rails direct vehicle wheelsets in an abrupt change of direction, making it highly desirable that wheels be rolling smoothly as they approach the switch. To best ensure that wheel flanges can be smoothly intercepted by switch point rails, tangent track should be placed immediately in front of the switch. The absolute minimum length of tangent track in advance of the point of the switch should be no less than 10 feet [3 meters], and much greater distances—33 to 50 feet [10 to 15 meters]—are desirable.

If a guarded curve is located in advance of the switch, the turnout should be positioned with the point of switch beyond the limits of the restraining rail. In situations where this is not possible, the restraining rail can be extended into the switch by use of a device variously known as a “cover guard” or “house top.” In such designs, the turnout is generally designed as a “fully guarded turnout.” See Article 6.5.5. for additional discussion of fully guarded turnouts.

Horizontal curves beyond the heel of the frog should generally be positioned beyond the last long tie of the turnout. In constrained sites, horizontal curves may begin on the long switch ties, but no closer than 20 inches [0.5 meters] from the heel joint of the frog. This distance allows room for tangent joint bars in bolted rail track or the thermite weld in all-welded installations. However, special, angled, rail seat tie plates may be required on timber switch ties. Custom concrete switch ties would definitely be required. In either case, the curve cannot have any superelevation.

If the following curve is guarded, and the restraining rail is on the frog side of the alignment, the curve should preferably be located so that the restraining rail terminates prior to the heel joint of the frog. If this is not possible without truncating the guarding to close to the curve, the restraining rail should extend into the frog and be continuous with the frog wing rail to provide continuous guarding action. Similarly, if the restraining rail is on the same side as the frog guard rail, the designer should consider extending the restraining rail all the way past the frog.

The non-standard layouts described should be avoided if possible; however, they will often be necessary within constrained areas, such as light rail vehicle (LRV) storage yards, so that adjoining curves need not be at (or below) the desirable minimum radius. While customized special trackwork will cost more, a life cycle analysis will often demonstrate that such trackwork will reduce the maintenance costs for the adjoining curved track appreciably.

6.3.1.2 Turnouts on Horizontal Curves

Turnouts can be constructed within curved track in difficult alignment conditions. Railroad operating personnel will state, however, that turnouts on curves provide a poor-quality ride. Track maintenance personnel contend that the curved turnouts consume a disproportionate amount of their maintenance budgets. Therefore, turnouts and crossovers should only be located in horizontally tangent track, except under the most unusual and constrained conditions. This will ensure that the track geometry through the special trackwork unit will be as uniform as possible, thereby improving wheel tracking and extending the life of both the special trackwork unit and the vehicle that operates over it. Note that if the main track curve is superelevated, the diverging track must also be superelevated. In the case of a turnout to the outside of a curve, this would create negative actual superelevation in the turnout curve, an undesirable condition that would actually be prohibited under a proposed (2010) revision to the FRA Track Safety Standards.

A turnout on a curve must be custom designed, and both the switch and the frog will be non-standard items. The design objective should be to provide an alignment that is as smooth and uniform as possible. Designers should note that the turnout geometry will differ appreciably from ordinary lateral turnouts located along tangent track. Parameters such as turnout lead distance and closure rail offsets will be distinctly different from those of a standard lateral turnout with the same frog number. Several good books exist on the subject, including Allen's *Railroad Curves & Earthwork*.[4]

6.3.1.3 Track Crossings on Curves

Either one or both tracks of a crossing (diamond) may be located in horizontally curved track if required by the selected alignment. This is often a requirement at a route junction. At such locations, it is typically allowable to have one or both sides of the track crossing on a curved alignment. In general, however, curved crossings should be avoided because they are typically

one-of-a-kind units and hence very expensive to procure, maintain, and ultimately replace. In addition, depending on the gradients of the intersecting tracks, the curved track may have adverse superelevation. This has a detrimental impact on the operation of trains over curved track.

6.3.1.4 Superelevation in Special Trackwork

In general, superelevation should not be used within any turnout, crossover, or track crossing, even if the main track is located on a curve. The correct amount of superelevation for one hand of the turnout will be incorrect for the other and an excessive underbalance or overbalance could result. A particularly dangerous situation occurs with a turnout to the outside of the curve, where a severe negative superelevation situation could be created on the diverging track. In ballasted track, normal deterioration of the track surface could quickly result in the diverging track becoming operationally unsafe.

When a superelevated curve is required beyond the frog of a ballasted track turnout, the superelevation should begin beyond the last long tie. In an otherwise intractable situation, superelevation could begin on the long ties by utilizing special plates or concrete switch ties to elevate one rail and rotate both rails; however, this is not recommended. In a direct fixation or embedded track turnout, superelevation can physically begin earlier, although typically not within 20 inches [500 millimeters] of the heel joint of the frog.

6.3.2 Vertical Track Geometry Restrictions

Turnouts, crossovers, and track crossings should be located on tangent profile grades whenever possible. This is because the critical portions of a turnout—the switch and the frog—are too rigid to conform to a vertical curve, which will cause the switch points to bind. The area between the switch and the frog can theoretically be curved vertically, but this practice is discouraged since ordinary construction tolerances make it difficult to confine the curvature to the closure rail area. Vertical track curvature outside of the turnout area should also be restricted; the absolute minimum distance from the switch and frog will depend on the type of track structure. In the case of ballasted track, for example, it is not practical to introduce any vertical curvature until after the last long tie of the turnout.

In difficult alignment conditions, vertical curvature at or near a turnout location may be necessary. If it is not possible to avoid a vertical curve within a turnout, every effort should be made to avoid non-standard track components, such as switch point rails or frogs, which must be shop-fabricated with a vertical curve. Generally, special designs can be avoided only if the middle ordinate of the vertical curve in the length of any switch point rail or frog is less than about 1/16 inch [1 mm].

Careful consideration must be given to track gradients at track crossings. The four frogs of the diamond must sit in a plane surface. If the profile of one track is on a significant grade, that will fix the elevations of the frogs and hence dictate the profile of the other track. Because of this, coordinating only the track centerline profiles of the intersecting tracks can be very misleading. The track alignment designer must instead analyze the gradients of each of the intersecting rails through to at least the ends of the frog arms and thereby verify that the diamond special trackwork is actually possible to fabricate and install. If the track gradients involved are steep,

one or both tracks may be significantly out of cross-level and that must be considered with respect to operating speed and track twist.

6.3.3 Track Design Restrictions on Location of Special Trackwork

While special trackwork can be required in ballasted, direct fixation, and embedded track sections, ballasted track turnouts are generally the most economical to procure and construct. Alignment design should minimize special trackwork requirements in direct fixation and embedded track environments because these elements are more expensive to procure and construct. Exceptions can be made, for example, when route geometry forces a particularly complex special trackwork layout with multiple turnouts and track crossings. It is often particularly difficult to design a satisfactory switch tie layout under such complex layouts and even more difficult to renew defective switch ties during subsequent maintenance cycles. In such special circumstances, the use of direct fixation special trackwork track may be preferable to a ballasted configuration.

On the other hand, direct fixation and embedded turnouts, once installed, generally require less maintenance than ballasted track specialwork. However, when embedded turnouts are life expired and require replacement, their renewal will generally be much more disruptive of transit operations than ballasted track renewal. Renewal of direct fixation specialwork can be comparatively simple, provided the plinth concrete is sound and the rail fastener anchorage locations do not need to be changed.

Yard trackage, which is usually ballasted, often requires that successive turnouts be constructed close to each other. The track designer should verify that turnouts are sufficiently spaced to permit standard switch ties to be installed and to permit maintenance personnel to renew individual switch ties. When special switch tie arrangements are required, the track designer should either detail the tie layout or require the track fabricator to provide a submittal of the proposed layout. In the latter case, the track designers should be certain ahead of time that a workable tie layout is possible. It is absolutely essential that switch ties supporting switches are perpendicular to the straight track. This is a problem when switches are placed immediately beyond a frog on the curved side of a turnout.

Special trackwork in embedded track can be particularly complicated and should be minimized. Route intersections within street intersections can be phenomenally complex and require intricate designs. When special trackwork must be located in embedded track, it should be positioned so that pedestrians are not exposed to switch point rails, and switch operating mechanisms and frogs are not positioned in pedestrian paths. See Chapter 12 for additional discussion on this topic. Switch operating mechanisms for embedded track turnouts are also difficult to procure and maintain, as noted in Article 6.3.4.2.

6.3.4 Interdisciplinary Restrictions on Location of Special Trackwork

Special trackwork should be located so as to minimize requirements for a special overhead contact system (OCS), sometimes referred to as a catenary system, or train control/signaling system structures and devices.

6.3.4.1 Overhead Contact System (Catenary) Interface

The installation of the overhead contact wire system (OCS) is complicated by the presence of turnouts and crossovers. Additional wires, pull-off poles, and insulating sections are needed to provide a smooth contact for the current collection device, regardless of whether it is a pantograph, trolley pole, or bow collector. Electrically isolating the opposite-bound main tracks is particularly difficult at double crossovers if the adjacent tracks are close together. These conditions should be discussed with the OCS designer to ensure that the catenary can be economically constructed and does not result in an unacceptable, visually intrusive installation in sensitive areas.

6.3.4.2 Train Control/Signaling Interface

Power switch machines for ordinary open track turnouts (ballasted or direct fixation) will typically be the same as those on freight railroad track and will usually fully comply with AREMA requirements. Interface details for these situations are available from vendors, and, once the train control system designers have identified the specific equipment they will be using, the track designer should have no particular design issues with the track side of the interface. Special attention is required to the configuration of the headblock ties in ballasted track and the plinth layouts for direct fixation special trackwork so as to match the selected switch machine and associated train control accessories.

Embedded turnouts are a different situation. As of this writing, there are only three vendors of power switch machines offering switch machines for embedded track turnouts in North America. None of these machines fully comply with AREMA requirements for switch machines. The principal problem is that switch locking is required by AREMA to allow automatic routing at design track speed so as to prevent any chance that the switch might be thrown under a train. However, presently available embedded switch machines do not provide locking as that term is defined by AREMA. Because of this shortcoming, many rail transit systems require train operators to stop at any turnout that is not equipped with a locking switch device, visually confirm switch point position, and only then proceed at a restricted speed. This causes delays and, for this reason alone, designers are strongly encouraged to avoid embedded turnouts whenever possible.

In addition, when embedded track switches are located in a lane shared with motor vehicles, inspection and maintenance is made appreciably more difficult since a flagman is absolutely necessary. The maintenance issue on embedded switches is compounded by being exposed to storm water runoff that washes street dirt into the track flangeways that eventually flows to the innards of the switch, requiring more frequent cleaning and repair than open track switches.

Signal system track circuits that are needed to determine track occupancy are more difficult to install and maintain in embedded track since the embedment material will restrict access to key areas where unintended shunts can cause signals to drop.

Train movements through embedded track turnouts, particularly those in mixed traffic lanes, will often be governed by traffic signals that are controlling not only rail movements but also motor vehicles running on rubber tires. Since the latter cannot be detected by track circuits, the LRT operator must be visually alert for motor vehicles (and pedestrians) that may be on conflicting paths even when he or she has a clear signal. It therefore should be possible for him to be visually alert for conflicting train movements as well, such as an LRV on an intersecting path that

began its movement on a clear traffic signal but which, for whatever reason, had not completed its movement prior to the traffic signal cycling. This is, of course identical to how any traffic intersection functions in the absence of LRT and how in-street junctions are handled on legacy streetcar lines. Implementing a similar procedure on new LRT routes could significantly simplify requirements for track circuits and associated insulated joints in embedded track.

Insulated rail joints in special trackwork can be especially complicated, particularly if they must be located in guarded track, in and around crossing diamonds, or within embedded track. Insulated rail joints in embedded track can be particularly problematic since the dirt and grime inherent in any pavement surface can result in shunting of signal current around the joint. The trackwork designer should coordinate with the signal designers to verify that a workable insulated joint layout is possible. In many cases, a workable track plan cannot be properly signaled, and the route geometry must be redesigned.

6.3.5 Miscellaneous Restrictions on Location of Special Trackwork

6.3.5.1 Construction Restrictions

The construction or contract limits of any trackwork contract should not be located within any special trackwork unit or in a segment of curved track. This will ensure that one contractor will be responsible for the uniformity of the horizontal and vertical track alignment through the special trackwork unit. (For similar reasons, neither contract limits nor limits-of-work should occur within a segment of horizontally or vertically curved track.) Often times such work limits are set by project administrative personnel without any understanding of the technical issues involved. The track designer should review those limits as early as possible in the project and request revisions if appropriate.

Whichever track constructor arrives at the interface second will generally need to cross the nominal “contract limit” so as to continue, complete, and confirm the final connections. This is customary and presents no particular issues provided the geographic limit of the work and the interface responsibilities are clearly spelled out in the construction contract documents. Often, a staging drawing clearly defining the designer’s intentions will be included in the bid documents. Construction staging is sometimes a discussion point in a project Basis of Design Report (BODR) but those discussions will not be binding on the contractors unless the BODR is included in the contract documents; something that rarely occurs.

It is not uncommon to have an elevation bust between the two contracts based on benchmark discrepancies or survey misunderstandings. This type of interface understanding must be confirmed early in the design stage and then early in the construction stage by both contractors.

6.3.5.2 Clearance Restrictions

Special trackwork should be located with adequate clearances from fixed trackside obstructions. For example, unless the vehicles are equipped with automatic bridge plates for passenger access, tangent track or track with a large curve radius is required alongside station platforms to meet the tight (platform to vehicle floor gap) tolerances required by ADAAG. If a station platform is located ahead of a point of switch, the minimum tangent track distance between the end of the platform and the point of switch should be equal to the truck center length of the light rail vehicle (LRV) plus the car body end overhang. The lateral position of switch stands and the height of

switch machines above the top of rail are also considered in clearances of trackside obstructions. Refer to Chapter 3 for additional design guidance on special trackwork clearances.

6.3.5.3 High Volume of Diverging or Converging Movements

Track designers should be very cautious whenever the route geometry results in a preponderance of the traffic passing through the curved side of a turnout. High traffic volumes through the curved side of a switch will result in accelerated wear of the switch point rail and the adjacent stock rail. Whenever possible, turnouts at junctions should be oriented to guide the branch with the more frequent or heavier traffic over the straight part of the turnout. If the traffic is (or will eventually be) approximately equal, consideration should be given to an equilateral turnout design as discussed in Article 6.4.4. This will reduce wear and associated maintenance of the switch points. Other non-conventional turnout layouts can be used to give the diverging movement the straight side of the switch. **Figure 6.3.1** illustrates such a turnout on a European LRT system. Selective use of such non-standard special trackwork details can often resolve a problematic alignment issue at relatively low cost.

Figure 6.3.1 Right-hand turnout with a left-hand switch

Turnouts at the end of a double-track segment should be oriented to guide the facing point movement over the straight side of the turnout. If this orientation results in an unsatisfactory operating speed for the trailing movement, the designer should consider using either an equilateral turnout design or a turnout with a flatter divergence angle and curve than might ordinarily be provided. Ordinarily, facing point diverging movements should be limited to situations where the single-track section is temporary and the double-track section is to be extended.

6.3.5.4 Track Stiffness

Ballasted turnouts, crossovers, and crossing diamonds have a considerably higher track modulus than ordinary ballasted track due to their mass and the frequent interconnections between rails. Nevertheless, ballasted specialwork with conventional rail fastenings of limited resilience is somewhat more resilient than either direct fixation or embedded specialwork layouts. Because of this differential, ballasted track turnouts located close to interfaces with stiffer track structures will provide a poor-quality ride and require more frequent track surfacing, particularly if vehicle speeds are relatively high. To avoid these circumstances, main tracks where vehicles operate at speeds greater than 45 mph [70 km/h] should not have specialwork units located within 250 feet [75 meters] of a transition zone between ballasted track and a more rigid track structure. As a

guideline, this distance can be reduced in areas where modest operating speeds are contemplated. A minimum travel time of 3 to 5 seconds between the special trackwork unit and a more rigid structure is recommended. Design exceptions will require stiffening of the ballasted track or retrofitting of the adjoining track to be more resilient.

6.3.5.5 Noise and Vibration Issues

Even well-designed special trackwork will be a source of noise and vibration. As such, special trackwork installations are undesirable in the vicinity of residential buildings, schools, hospitals, concert halls, and other sensitive noise and vibration receptors. If special trackwork must be located in such areas, investigation of possible noise and vibration mitigation measures should be undertaken. Such investigations should also include the ramifications of repositioning the special trackwork away from the area of concern.

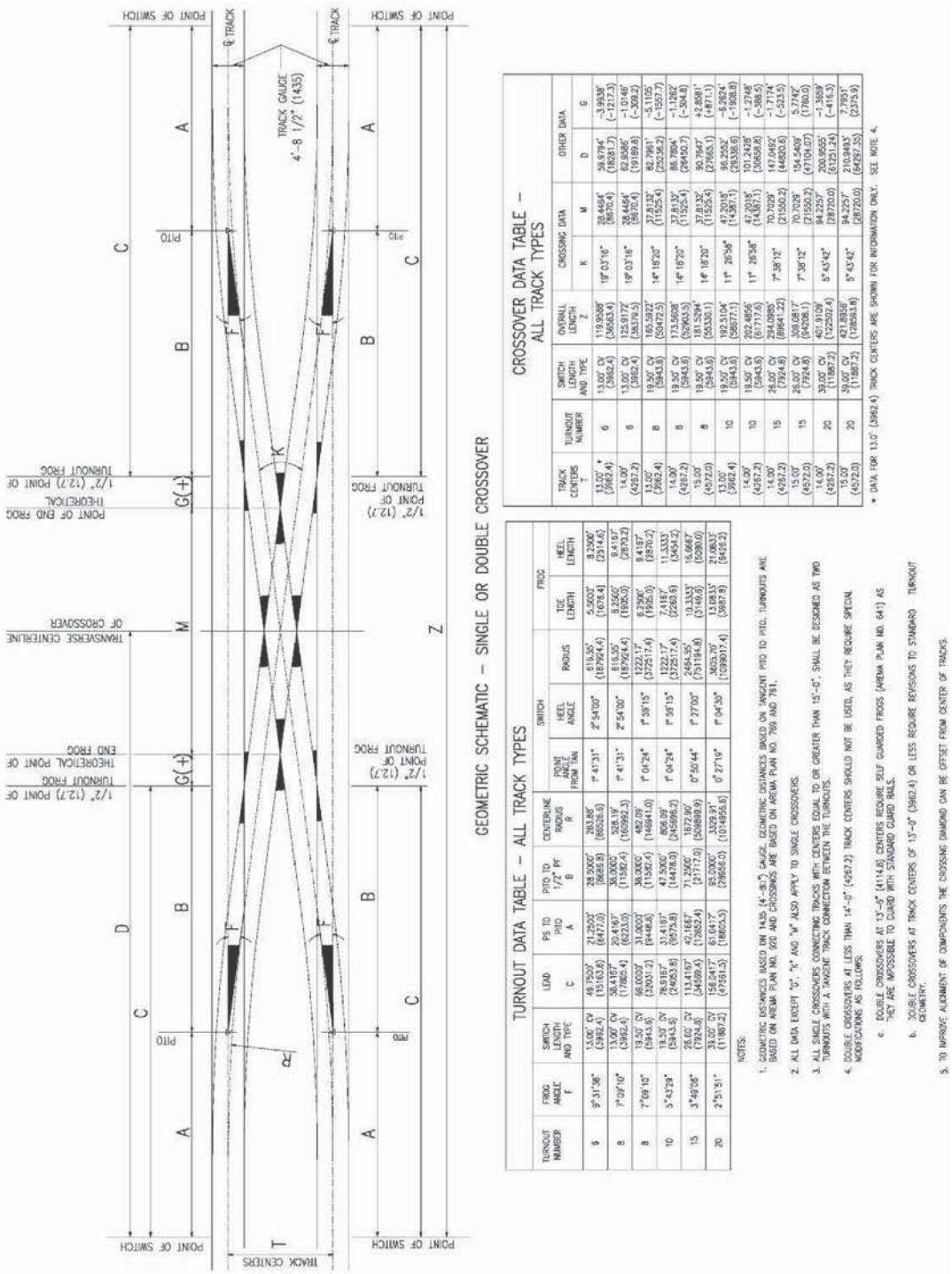
6.4 TURNOUT SIZE SELECTION

Track designers have a wide array of standard turnout geometric configurations to choose from when considering route alignment. While not all transit systems can use the same menu of turnouts and crossovers, the designer can usually achieve an acceptable route alignment without resorting to special turnout designs. Using standard, off-the-shelf, and service-proven materials will reduce the probability that future maintenance will be complicated by the need to purchase expensive one-of-a-kind products. Using standard materials also prevents a situation in which essential replacement parts may not be available when needed. **Figures 6.4.1 to 6.4.4** show the common sizes of turnouts and crossovers on timber ties with railbound manganese frogs. See Figures 6.9.1 through 6.9.3 for illustrations of similar turnouts on concrete ties and with solid manganese steel frogs.

Situations will arise when a non-standard turnout design is needed. In such cases, justification should be documented. This validation should include the reasons why a particular turnout size is required; what alternatives were investigated; why standard options were unacceptable; and the ramifications of using a smaller turnout, including its effect on vehicle operations, signaling systems, and OCS systems. Consideration should also be given to procurement of a spare assembly along with the original unit, so as to save the design and tooling costs that would be incurred to purchase a replacement unit at a later date. This provides an immediate replacement part if one is needed in an emergency situation.

6.4.1 Diverging Speed Criteria

Turnout size (by either frog number or radius) should be selected to provide the highest diverging movement speed possible that is consistent with adjoining track geometry. A high speed turnout is not needed if the adjoining track geometry restricts operating speed. Similarly, a sharp turnout should generally not be used in a track segment that has no restrictions on operating speed.



TURNOUT DATA TABLE – ALL TRACK TYPES

TURNOUT NUMBER	FROG ANGLE F	SWITCH LENGTH AND TYPE	LEAD C	PS TO PITO A	PITO TO 1/2" PF B	CENTERLINE RADIUS R	SWITCH POINT ANGLE FROM TAN	SWITCH HEEL ANGLE	SWITCH RADIUS	FROG TOE LENGTH	FROG HEEL LENGTH
6	9°-31'36"	13.00' CV (3962.4)	49.7500' (15163.8)	21.2500' (6477.0)	28.5000' (8685.8)	283.86' (86526.6)	1°-41'31"	2°-54'00"	816.55' (187924.4)	5.5600' (1678.4)	8.2500' (2514.6)
8	7°-09'10"	13.00' CV (3962.4)	58.4167' (17806.4)	20.4167' (6223.0)	38.0000' (11582.4)	328.19' (100992.3)	1°-41'31"	2°-54'00"	816.55' (187924.4)	8.2500' (1935.0)	8.4167' (2870.2)
8	7°-09'10"	19.50' CV (5943.6)	68.0000' (20001.2)	31.0000' (9448.6)	38.0000' (11582.4)	482.09' (146941.0)	1°-04'24"	1°-59'15"	1222.17' (372517.4)	8.2500' (1935.0)	8.4167' (2870.2)
10	5°-43'29"	19.50' CV (5943.5)	78.9167' (24053.8)	31.4167' (9575.8)	47.5000' (14478.0)	806.09' (245698.2)	1°-04'24"	1°-59'15"	1222.17' (372517.4)	7.4167' (2260.6)	11.3333' (3454.2)
15	3°-49'06"	26.00' CV (7924.8)	113.4167' (34589.4)	42.1667' (12852.4)	71.2500' (21717.0)	1672.90' (509899.9)	0°-50'44"	1°-27'00"	2464.55' (751194.8)	10.3333' (3149.9)	16.6667' (5080.0)
20	2°-51'51"	39.00' CV (11887.2)	158.0417' (47591.5)	61.0417' (18605.5)	95.0000' (28956.0)	3329.91' (1014956.6)	0°-27'19"	1°-04'30"	3605.20' (1099017.4)	13.0833' (3987.8)	21.0833' (6425.2)

NOTES:

1. GEOMETRIC DISTANCES BASED ON 14.35 (4'-8½") GAUGE. GEOMETRIC DISTANCES BASED ON TANGENT PITO TO PITO. TURNOUTS ARE BASED ON AREMA PLAN NO. 600 AND CROSSINGS ARE BASED ON AREMA PLAN NO. 769 AND 791.
2. ALL DATA EXCEPT "D", "K" AND "M" ALSO APPLY TO SINGLE CROSSOVERS.
3. ALL SINGLE CROSSOVERS CONNECTING TRACKS WITH CENTERS EQUAL TO OR GREATER THAN 15'-0", SHALL BE DESIGNED AS TWO TURNOUTS WITH A TANGENT TRACK CONNECTION BETWEEN THE TURNOUTS.
4. DOUBLE CROSSOVERS AT LESS THAN 14'-0" (4267.2) TRACK CENTERS SHOULD NOT BE USED, AS THEY REQUIRE SPECIAL MODIFICATIONS AS FOLLOWS:
   a. DOUBLE CROSSOVERS AT 13'-5" (4114.8) CENTERS REQUIRE SELF GUARDED FROGS (AREMA PLAN NO. 641) AS THEY ARE IMPOSSIBLE TO GUARD WITH STANDARD GUARD RAILS.
   b. DOUBLE CROSSOVERS AT TRACK CENTERS OF 13'-0" (3962.4) OR LESS REQUIRE REVISIONS TO STANDARD TURNOUT GEOMETRY.
5. TO IMPROVE ALIGNMENT OF COMPONENTS THE CROSSING DIAMOND CAN BE OFFSET FROM CENTER OF TRACKS.

CROSSOVER DATA TABLE – ALL TRACK TYPES

TRACK CENTERS T	TURNOUT NUMBER	SWITCH LENGTH AND TYPE	OVERALL LENGTH Z	CROSSING DATA K	CROSSING DATA M	OTHER DATA D	OTHER DATA G
13.00' * (3962.4)	6	13.00' CV (3962.4)	119.9586' (36563.4)	19°-03'16"	28.4464' (8670.4)	59.9794' (18281.7)	-3.9938' (-1217.3)
14.00' (4267.2)	6	13.00' CV (3962.4)	125.9172' (38379.5)	19°-03'16"	28.4464' (8670.4)	62.9586' (19189.8)	-1.0146' (-309.2)
13.00' (3962.4)	8	19.50' CV (5943.6)	165.5922' (50472.5)	14°-18'20"	37.8133' (11525.4)	82.7961' (25236.2)	-5.1105' (-1557.7)
14.00' (4267.2)	8	19.50' CV (5943.6)	173.5608' (52903.5)	14°-18'20"	37.8133' (11525.4)	86.7804' (26450.7)	-1.1282' (-304.8)
15.00' (4572.0)	8	19.50' CV (5943.6)	181.5294' (55330.1)	14°-18'20"	37.8133' (11525.4)	90.7647' (27665.1)	42.8581' (+871.1)
13.00' (3962.4)	10	19.50' CV (5943.6)	192.5104' (58677.1)	11°-26'56"	47.2018' (14387.1)	96.2552' (29338.6)	-6.2624' (-1908.8)
14.00' (4267.2)	10	19.50' CV (5943.6)	202.4856' (61717.6)	11°-26'56"	47.2018' (14387.1)	101.2428' (30858.8)	-1.2748' (-388.5)
14.00' (4267.2)	15	26.00' CV (7924.8)	294.0865' (89641.22)	7°-38'12"	70.7029' (21550.2)	147.0492' (44820.6)	-1.7174' (-523.5)
15.0' (4572.0)	15	26.00' CV (7924.8)	309.0817' (94208.1)	7°-38'12"	70.7029' (21550.2)	154.5409' (47104.07)	5.7742' (1760.0)
14.00' (4267.2)	20	39.00' CV (11887.2)	401.9109' (122502.4)	5°-43'42"	94.2257' (28720.0)	200.9555' (61251.24)	-1.3559' (-413.3)
15.00' (4572.0)	20	39.00' CV (11887.2)	421.8858' (128593.6)	5°-43'42"	94.2257' (28720.0)	210.9433' (64297.35)	7.7951' (2375.9)

* DATA FOR 13.0' (3962.4) TRACK CENTERS ARE SHOWN FOR INFORMATION ONLY. SEE NOTE 4.

Figure 6.4.1 Turnout and crossover data

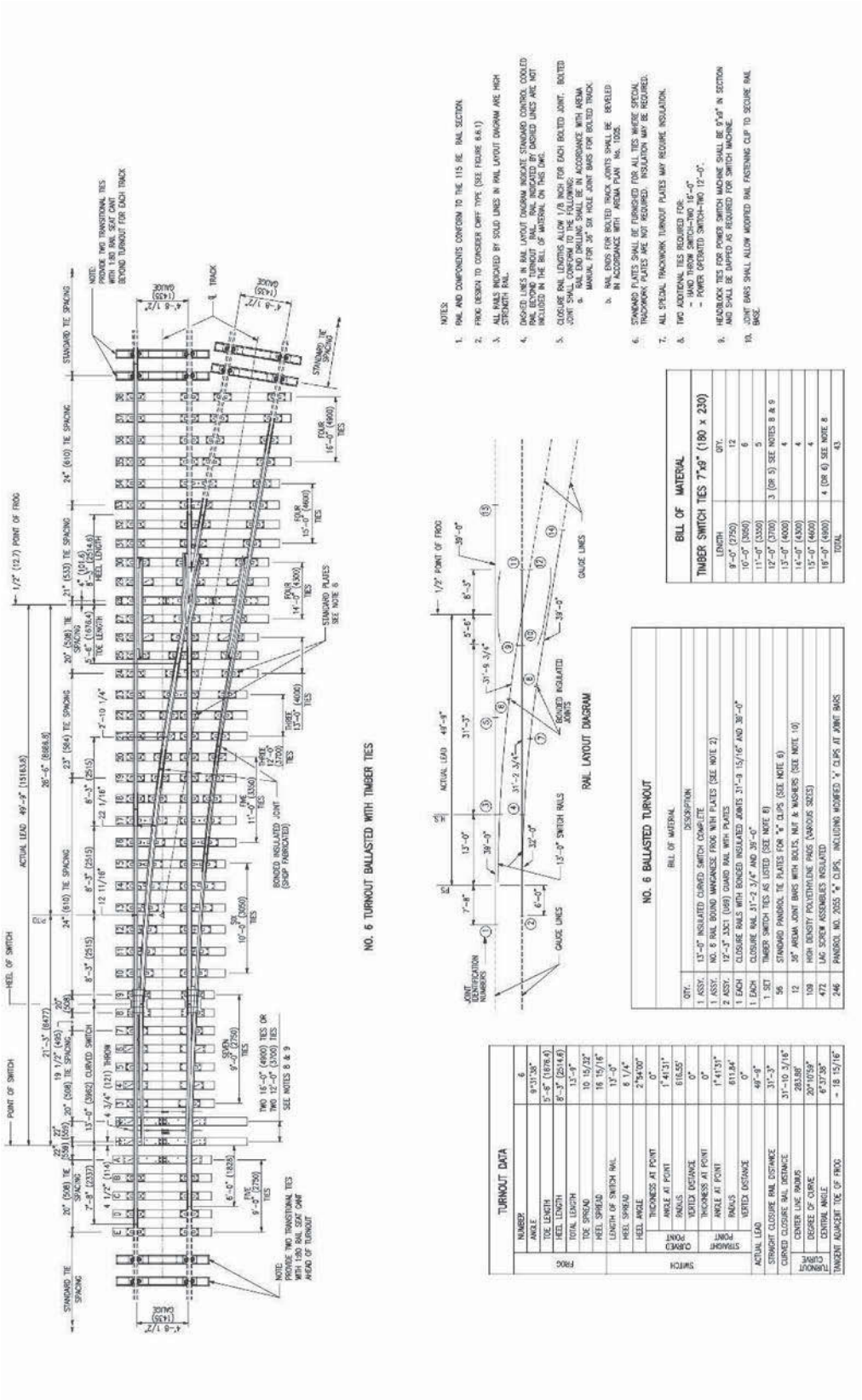


Figure 6.4.2 No. 6 turnout—ballasted timber ties with 13' curved switch

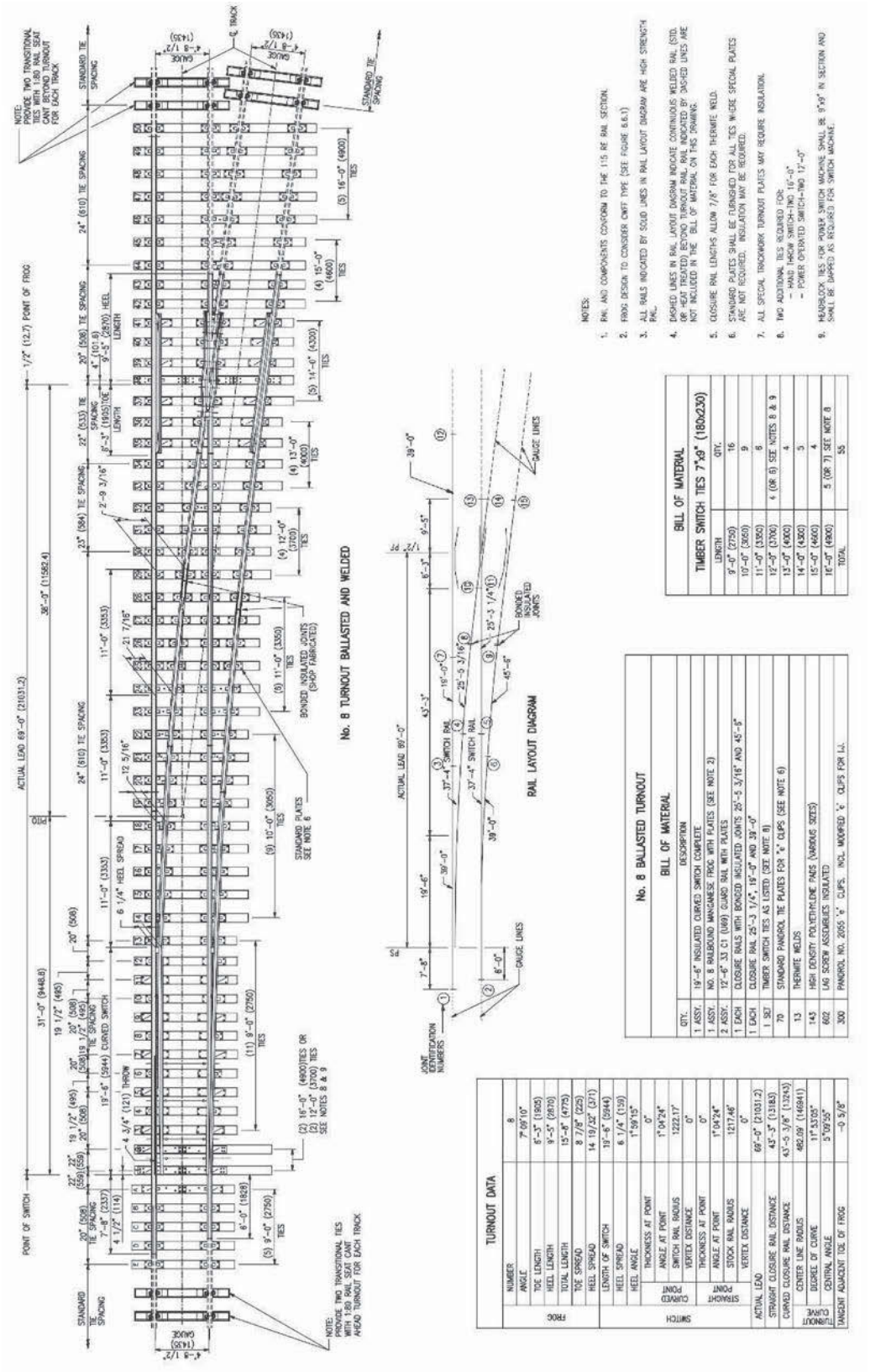


Figure 6.4.3 No. 8 turnout—ballasted timber ties with 19'6" curved switch

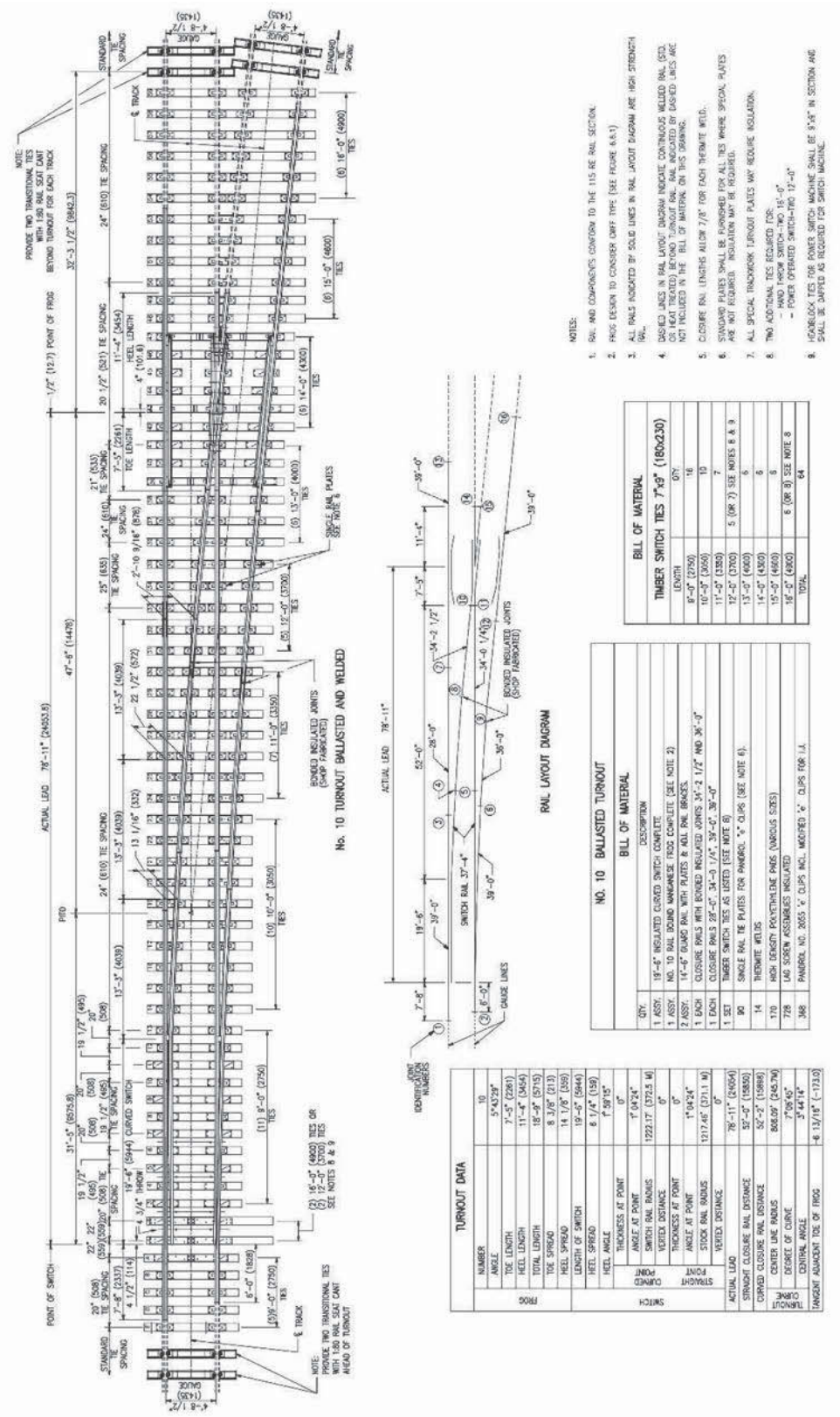


Figure 6.4.4 No. 10 turnout—ballasted timber ties with 19’6” curved switch

Limits on operating speeds through the curved side of turnouts are typically based on the turnout geometry and the maximum unbalanced superelevation criteria adopted for the system. In many cases, the closure rail radius zone will impose a greater restriction on operating speed than the switch radius, particularly if tangential switch geometry is not used. There are typically no operating speed restrictions on the straight through side of a turnout; however, if the turnout is a lower number, but the operating speed through the straight side is high, it may be appropriate to use a longer frog guard rail on the straight side of the frog than might ordinarily be called for.

While higher number/radius turnouts will generally have higher initial costs, they will incur less wear and tear and can be more economical in the long run. There are reasonable limits to this rule, of course—it makes little sense, for example, to install a No. 20 turnout that will never be traversed at more than 25 mph [40 km/hr]. In general, trackwork designers will find that No. 8, No. 10, and possibly No. 15 turnouts will be the most economical choices for main line track on virtually any light rail system.

When selecting turnout sizes, other issues may dictate the choices. The menu of turnouts need not use all even-numbered or all odd-numbered frog angles; these can be mixed for project-specific reasons. In the case of a project that will share track with a freight railroad that uses odd-numbered turnouts, such as No. 9 and No. 11 sizes, it would make little sense for LRT-only turnouts on the same project to be even-numbered frogs. In such cases, the standards of the organization that will actually own and maintain the tracks should be given precedence.

6.4.2 Turnout Size Selection Guidelines

The following criteria recommend various turnout sizes for various track applications. The typical conditions and operating speed objectives are based on an old “rule of thumb” that stated that the frog number should be about one-half of the desired diverging movement operating speed in miles per hour [roughly one-third of the desired speed in kilometers per hour]. Handbook users should keep in mind that operating speed objectives vary among light rail operations, as well as from one portion of an LRT system to another. “High speed” on one LRT system may be considered “low” on another. Streetcar projects often need to vary radically from these guidelines. Accordingly, the recommendations that follow should be modified to suit project-specific requirements:

- Route junctions between primary tracks should use No. 15 turnouts. An even larger number turnout might be considered if the route geometry in proximity to the turnout does not restrict higher speed operations. When sufficient space is not available for a No. 15 turnout, or if there are nearby speed restrictions (such as station stops or an at-grade roadway crossing), a sharper turnout, such as a No. 10, may be considered.
- No. 10 turnouts should typically be used for terminal station crossover tracks and connections between primary main line tracks and slower speed yard and secondary tracks, including center pocket tracks used for turnbacks at intermediate points. When design space for a No. 10 turnout is not available, a No. 8 turnout may be sufficient. If the LRT line is likely to be extended beyond some initial terminal station in the not-too-distant future, but the crossovers will remain for emergency use, it may be satisfactory to use lower numbered turnouts at the interim terminal.

- Seldom-used crossover tracks that are provided for emergency and maintenance use only should use No. 8 turnouts. When sufficient design space for a No. 8 turnout is not available, a No. 6 turnout may be considered.
- Turnouts within maintenance facilities and storage yards should use either No. 8 or No. 6 turnouts. If turnouts sharper than a No. 6 are required because of a constrained site, consideration should be given to use of a curved frog turnout. Main line connections to the maintenance facility and storage yard should generally use No. 8 or No 10 turnouts, the choice being somewhat dependent on any curve speed restrictions that might occur beyond the turnout on the yard lead track wye arrangements.
- Turnouts that are located in embedded track are often in odd geometric layouts and thus must be sized in accordance with the use and function of the turnout. Alternatives to the use of an embedded turnout should always be investigated.

The recommendations provided above are based on the use of even-numbered frogs for turnouts sharper than a No. 15 turnout. There is nothing inherently superior about the use of even-numbered frogs. Virtually all railroads west of the Mississippi River long ago standardized odd-numbered frogs with No. 7, No. 9, and No. 11 turnouts being common. The use of odd-numbered turnouts on LRT is perfectly legitimate and may have some advantage on projects that share track with a western freight carrier. However, note that the freight railroad's standard rail section is very likely heavier than an LRT's standard rail section, so interchangeability of parts will be very limited.

In general, it is highly desirable to use no more than three or four sizes of turnout on an LRT system so as to limit maintenance inventory and simplify the job of the track maintenance staff. If there is a perceived "need" for only one or two of a particular turnout size, it is usually better to redesign the track geometry so as to use some other turnout that is used elsewhere on the project in greater numbers. Systems that include joint track operation with a freight railroad should closely consider the turnout standards of the freight carrier, but not necessarily let that dictate the size of turnouts to industrial sidetracks. The governing factor should be the preferences of the entity who will actually maintain the turnout. For example, if the freight railroad's preference for an industrial sidetrack is a No. 9 turnout, but the LRT project standards include No. 8 turnouts and No. 10 turnouts and the turnout in the shared main track will be maintained by the LRT agency, the turnout should conform to the agency's standards. Turnouts that are not in the LRT route main track and will be used only by the freight railroad and maintained by their forces should conform to the railroad's standards, including rail section.

6.4.3 Sharp Frog Angle/Tight Radius Turnouts

Many light rail systems, particularly legacy streetcar operations, use turnouts that are sharper than those discussed above. No. 4 and No. 5 straight angle frogs are not uncommon.

Many difficult alignment conditions may be resolved using turnouts that have a continuous curve through both the switch and the frog. Operationally, these turnouts will typically give much more satisfactory service than straight angle No. 5 or No. 4 turnouts because the closure curve radius will be continuous through the frog. This makes it possible to achieve greater angles of divergence in a shorter lead distance while using a larger radius than would be possible with a

straight angle frog. The elimination of the short tangent through the frog will also eliminate the associated lateral jerk and provide a smoother ride.

Note that the radius of the curve can vary through the length of the turnout. Legacy streetcar systems often have embedded track turnouts where the switch has a relatively broad radius, such as 200 feet [61 meters]. Then, through a sequence of short compound curves, the radius is decreased to some much smaller value to match the main body of the curve. The curves through such turnouts are essentially a Searles spiral. The former ATEA had a set of standard spirals used for both curves and "branch-offs" (their term for a turnout) that effectively duplicated each other, making it relatively easy to add a switch at a street intersection.[5]

Some transit agencies have curved frog turnouts with radii as sharp as 50 feet [15.2 meters]. In virtually all cases, these sharp turnouts were required due to unique site conditions and the particular requirements of the system. While such sharp turnouts are not recommended for general application, there is nothing inherently wrong with their use provided that they meet the requirements of the transit operation, and the transit agency understands and accepts the limitations that sharp turnouts impose. Some of the restrictions imposed by sharp turnouts are the following:

- Vehicle fleet must be designed to be able to negotiate them. This may reduce the number of candidate light rail vehicles that can be considered for the system.
- Operations will be slower. Operating personnel must be made aware of the speed restrictions that the sharp turnouts impose and speed controls (signal systems, operating rules, or both) must be in place to restrict speeds to the allowable limit. This can be a significant problem on a system, or portion of a system, where vehicle speed is entirely under the operator's control. Most vehicle storage yard tracks, which are the most likely location for sharp turnouts, do not have signal systems that provide speed control. This makes it highly probable that sharp turnouts will be negotiated at higher-than-design speeds, leading to excessive wear, more frequent maintenance, and an increased risk of derailments. A common problem in this regard, known as "cracking the whip," is a distressingly common operating practice on many systems where the LRV operator may enter the turnout at the posted speed limit but then accelerate. The result is that the rear truck of the LRV enters the curve and travels through the turnout at a much higher speed than intended. High rail and wheel wear will occur, resulting in derailments of rear trucks. The problem can be even more severe when the trailing LRVs in a multi-car train travel even faster through the turnout. Cab signaling systems, which prevent the train operator from exceeding a signal speed command, can alleviate this problem.
- Maintenance expenses will be higher. Even if vehicle speed is controlled, either through the signal system or by strict enforcement of operating rules, sharp turnouts will incur more wear than flatter turnouts. If the associated maintenance expense is preferable to the additional first cost of procuring enough right-of-way to permit the use of flatter turnouts, then sharp turnouts may be a prudent choice. If, on the other hand, a life cycle cost analysis shows that procuring additional right-of-way that allows flatter turnouts will reduce the overall expense, then that course should be pursued.

6.4.4 Equilateral Turnouts

Equilateral turnouts split the frog angle in half between both sides of the turnout, producing two lateral diverging routes. Both sides of the turnout are curved. Equilateral turnouts are occasionally suggested for the end of double-track locations and for locations where a turnout must be installed on a curve. The track designer should consider the following characteristics:

- A perfectly symmetrical equilateral turnout will evenly divide the frog angle and the switch angle. The division of the switch angle will require a custom set of stock rails, each with half the normal stock rail bend. This arrangement is preferred when both hands are used in the facing point direction, such as the diverging turnout at a route junction.
- An alternative to customized stock rails is to configure the switch as if it would be in an ordinary lateral turnout, giving one movement the straight route through the switch and the other movement the lateral route. The frog does not need to be oriented symmetrically, and the optimum alignment for each route may be achieved by rotating it by an amount equal to the switch angle. This switch and frog orientation would be preferred for an end-of-double-track location where extension of the double track is not expected to occur in the near future.
- If the switch angle is to be split equally, curved switch point rails will need to be specially designed and fabricated since each point rail must not only have a concave curve on its gauge face, but also a concave vertical surface on its back face. Such switch point rails are not off-the-shelf items, and the transit system will have to procure and inventory spare switch point rails for future replacement. Straight switch point rails on the other hand, such as the AREMA 16’-6” [5,029-millimeter] design, can be obtained off the shelf although they still must be matched to custom stock rails. If the switch is oriented as in an ordinary lateral turnout, standard switch point rails can be used.
- The lead distance of the equilateral turnout need not have any direct correlation to the customary lead for a lateral turnout utilizing the same size of frog. The closure curves between the switch and frog can be configured to any geometry that is suitable to meet the speed objectives of the turnout.

Using an equilateral turnout to provide a turnout to the outside of a curve usually does not provide satisfactory ride quality and is, therefore, not recommended.

6.4.5 Curved Frogs

A straight frog is standard for most turnouts, for both normal and diverging train movements. Straight frogs generally provide satisfactory ride quality and have the advantage of being usable in both left- and right-hand turnouts, thereby reducing maintenance inventory. However, when the turnout is immediately followed by a curve in the same direction, the straight frog creates a “broken back curve” alignment. In lower numbered (sharp radius) turnouts, this condition will provide an undesirable ride quality. If a system will have a large number of low numbered turnouts, as is often the case for yard tracks, it may be beneficial to consider curved frogs that allow a uniform curve through the turnout and the track beyond. A better yard layout may be possible using curved frog turnouts, as shown in **Figure 6.4.5**, without incurring excessive costs. Curved frogs may be the only way a yard of sufficient capacity can be created on a constrained site.

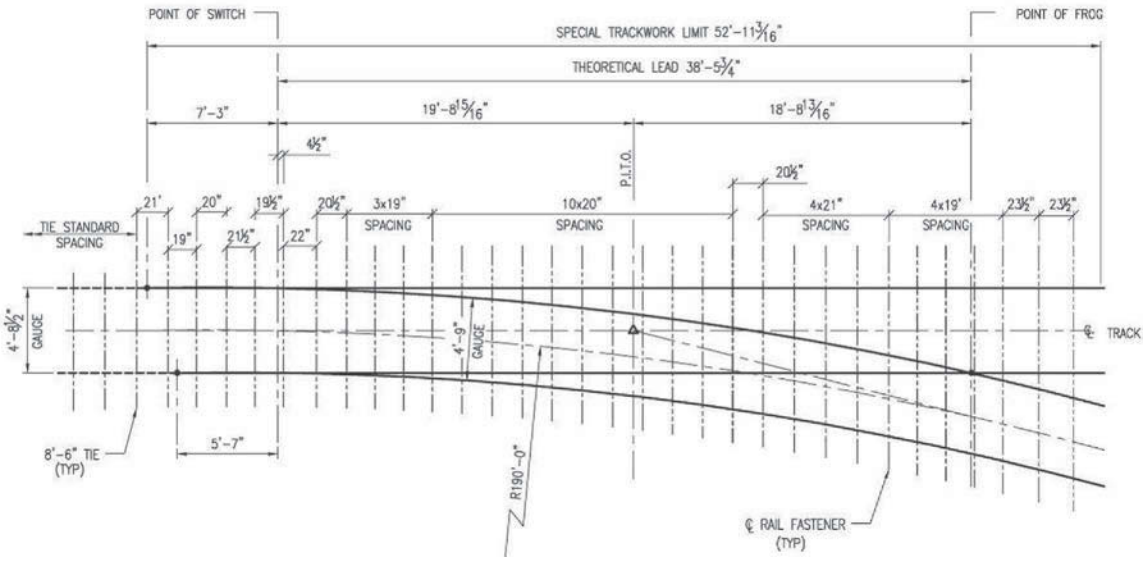
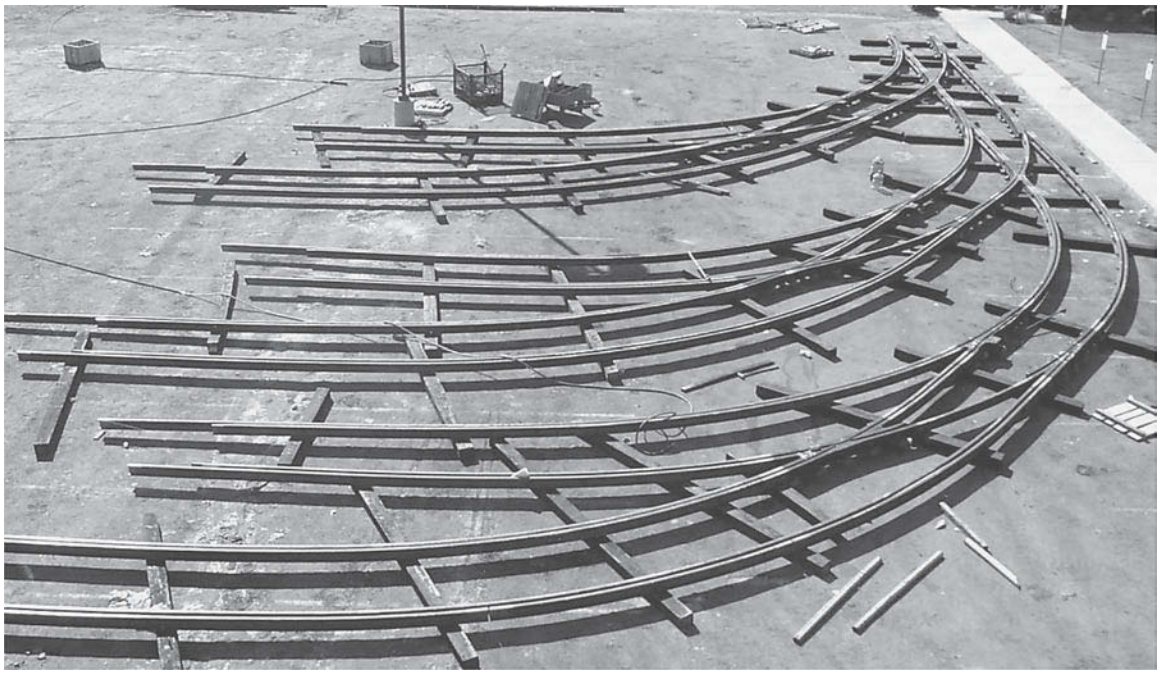


Figure 6.4.5 Typical curved frog turnout

A common yard track layout on streetcar projects is a ladder track that requires double curved frogs with both curves in the same direction, as shown in **Figure 6.4.6**. The flangeways and track gauges of such layouts must be carefully evaluated using Nytram plots so that wheels are not misdirected at the point of frog. Achieving exact gauge during construction is critical.



(Photo courtesy of Progress Rail Services)

Figure 6.4.6 Ladder track with double curved frogs

6.4.6 Slip Switches and Lapped Turnouts

Slip switches and lapped turnouts are often suggested as a means of concentrating a large number of train movements into a constrained site. Such components are very expensive to procure and maintain and are seldom justifiable in a life cycle cost analysis. They should only be considered in cases, such as yard tracks, where extremely restrictive rights-of-way leave no other design options.

6.4.7 Track Crossings (Diamonds)

Whenever possible, track crossings (diamonds) should have angles that do not require movable point design. Movable point crossings have high initial costs, require more frequent maintenance, and require a separate set of switch machines; therefore, these crossings should be used only as a last resort. To provide for the use of rigid crossings only, the route alignment engineer will be required to configure the tracks so that crossing tracks intersect at an angle at least equal to that of a No. 6 frog ($9^{o}31'38"$). Some systems have successfully used crossings with flatter angles, but these crossings are not recommended because of the increased potential of derailment at the unguarded center frog points. If a flat-angle movable point crossing appears to be required at a location such as a route junction, a detailed investigation of alternatives should be conducted before trackwork final design commences. These alternatives could include spreading track centers to permit one track to cross the other at a sharper angle or substituting a crossover track in advance of the junction for the crossing diamond. Simulations may be required to determine if the operational scenarios resulting from an alternative track plan are acceptable. The maintenance requirements of the baseline movable point crossing should be included in the analysis, including the operational restrictions that may be enforced during such maintenance.

6.5 SWITCH DESIGN

The switch area is the most critical portion of any turnout. Most turnout maintenance is switch related, requiring both trackwork and signal maintenance. Most derailments occur at and are caused by unmaintained or neglected switches. As such, switches are one of the most important locations at which to examine the interaction between the wheel and the rail. The following articles discuss the various types of switch designs that can be used on light rail systems and provide guidelines for selecting a design to implement.

6.5.1 Conventional Tee Rail Split Switches

Most rail transit systems in North America use switch point rails that are identical or similar to designs used by North American freight railroads. Such switches, known as split switches, generally conform to designs promulgated by the American Railway Engineering & Maintenance-of-Way Association (AREMA). Split switches are produced by first bending and then machine planing a piece of standard tee rail to create a knife edge point on one end. The sharpened point then lays up against a section of standard rail (the stock rail) and diverts the flanged wheel from one track to another. Split switches are relatively inexpensive to produce and provide satisfactory service under most operating scenarios.

For an ordinary lateral turnout, both split switch point rails can be straight or one can be straight and the other one curved. Straight switch point rails can be used universally with either right- or left-hand turnouts, but are almost always an inferior choice for a diverging route. As a guideline, curved switch point rails are recommended for all transit designs so as to provide a smooth transition into a turnout.

With the standardization of CWR and the elimination of high-maintenance rail joints, the conventional design of bolted heel blocks has been replaced with floating heel blocks. The floating heel block design eliminates the bolted connection at the heel of the switch rail. Instead, the switch rail extends beyond the nominal heel location and into the turnout's closure rail area. This makes it possible to thermite weld the switch point rail to the closure rails. This design appears to function best when the switch point rails are long enough to flex rather than pivot as with conventional bolted heel blocks. The floating heel block is connected only to the switch point rail and acts as a spacer or separation block when bearing against the web of the stock rail, thereby providing the designed heel of switch spread.

Switch point stops provide the proper spread between the point rail and the stock rail. The switch point stop supports the switch point rail against lateral wheel forces. If the stops do not bear against the stock rail web when the switch point rail is closed, lateral loads from the wheels will result in flexing of the point, possibly opening the switch point if sufficient slack is available in the throw rod connections. This opening could result in the next wheel "picking" the point, leading to a broken switch point or, possibly, an actual derailment.

Short switch rails, such as the AREMA 13' [3,962 mm] curved point design, cannot take full advantage of a floating heel block because the short length available for flexure would require excessive switch machine force to throw the switch. There are two options for relieving this issue:

- The flexive zone can be extended beyond the nominal length of the switch. For the nominal 13'-switch [3,962-meter] above, the flexive length might actually be 16 to 20 feet [5 to 6 meters]. As such, the spread dimension at the end of the actual flexive zone would be much greater than the customary 6 ¼ inches [159 mm] that will still exist at the nominal heel of the switch.

- A portion of the base of the switch rail straddling the nominal heel can be machined away on both sides, making the rail more flexible in that zone.

So as the ensure point rail movement with a minimum of throw force and also for simplicity of point rail change out, switches of 13 feet in length may best be detailed for jointed heel block design. As of 2010, AREMA had no floating heel standard although AREMA Committee 5, which is responsible for the *Portfolio of Trackwork Plans*, has it under discussion.

6.5.2 Uniform and Graduated Risers

Split switch designs, whether using conventional AREMA geometry or tangential alignment, typically elevate the top of the switch point rail approximately ¼ inch [6 millimeters] above the top of the stock rail. This prevents false flanges on worn wheels from contacting the top of the stock

rail and possibly lifting the wheel off the top of the switch rail. This raised point rail design is developed by having a lower stock rail seat elevation at each switch plate. This additional elevation can be eliminated once the switch rail has diverged sufficiently from the stock rail to eliminate the risk. The two design details that achieve this vertical transition are called “uniform risers” and “graduated risers.”

Uniform riser design switch point rails—which are the preferred design for transit work—use a series of elevation runoff switch plates located immediately beyond the heel of switch area. The runoff plates are designed to gradually lower the raised point rail to the elevation of the surrounding rails. The gradual decrease is obtained by incrementally lowering the base of the closure rail over a series of machined plates at $^{1}/_{16}$ inch [1.5 millimeters] per plate for three plates. If a more gradual decrease is desired, the incremental steps can be reduced to $^{1}/_{32}$ inch [1 millimeter], requiring seven runoff plates. This design requires no vertical bends in the switch point; hence, the bottom of the switch point is level and all of the riser plates beneath the switch point are the same elevation (i.e., “uniform”).

Graduated riser design switch point rails permit the decrease in switch point height to occur within the switch point length so the two rails are at the same level at the switch heel. This design requires somewhat abrupt reversed vertical bends to be applied near the heel of the switch point rail. In the vicinity of these bends, the thickness of the underlying switch plates are gradually reduced, hence the name “graduated riser.” For conventional railroad specialwork, graduated risers permitted the use of cheaper and simpler “hook twin plates” to fasten the rails beyond the heel block instead of the series of more expensive machined runoff plates.

The vertical bends in a graduated riser switch point are somewhat of a ride quality issue, especially for the straight movements through the turnout. In addition, the mating of the base of the switch rail to the base of the stock rail requires extra machining. Nonetheless, for freight railroads, the lower first cost of the hook twin plates made the graduated risers attractive for low-speed operations. However, hook twin plates are not used in transit work because it would be difficult to provide insulating elements for stray current isolation. Therefore, it is necessary to provide machined plates beyond the heel of the switch anyway. The extra machining necessary for running off the elevation of uniform riser switch rails is more economical than the vertical bends and additional milling required on a graduated riser switch point.

As a guideline, uniform risers will usually provide the best and most economical service for turnouts in main tracks and any turnout where insulation is required. Uniformity of maintenance suggests that switches in yard and secondary tracks on the same transit system should also use uniform risers even if they are not insulated. Graduated risers should only be considered for use in tracks where stray current isolation is not required and then only if the system does not include any uniform riser switches of the same length. The differences between a uniform riser switch point and a graduated riser switch point are not obvious at first glance. Stocking both designs in inventory raises the possibility that maintainers may inadvertently install the wrong design at a given location, resulting in poor performance and possible derailments.

Regardless of the switch point design selected, all switches perform best if there is a regular program of wheel truing maintenance to eliminate false flanges and the resultant battering of the stock rails. The standards for vehicle wheel maintenance play an important part in the switch

point design and must be considered when contemplating the interface between the wheel and switch point.

6.5.3 Tangential Geometry Switches

Conventional North American switch points require that wheels make a somewhat abrupt change of direction near the point of switch. The actual angle at the point rail will vary depending on the length from the switch point to the heel of switch, but it typically ranges between 1 and 3 degrees. Even if the diverging switch point is curved, it still intersects the gauge line of the straight stock rail at an angle that, while reduced from the angle at the switch heel, is still noticeable. (Such switches are known as secant switches, after the trigonometric terminology.) Depending on the speed of the transit vehicle through the switch, this change in direction can produce an uncomfortable ride since the jerk rate is effectively infinite for a very brief period.

In addition, a switch point used for diverging movement will frequently incur a much greater amount of wear due to the abrasive impact associated with redirecting the vehicle wheels. In the case of switches used for a converging movement, the straight stock rail will often incur severe lateral wear in the area immediately ahead of the point of switch where the truck in the steering position uses the stock rail to straighten out. Experience has shown that trucks with severe skewing will also wear the bent stock rail face before the truck attempts to align with the tangent track. This is a serious concern, as the gauge face of the stock rails provides the required protection for the switch point during facing movements. If the gauge face of the stock rail is worn back, the end of the switch point could be exposed to possible wheel impact.

To improve switch performance and service life, European track designers developed “tangential geometry” switches. In a tangential geometry switch, the switch point that deflects the diverging movement is not only curved but also oriented so that the curve is tangential to the stock rail. The wheel is not required to make an abrupt change of direction; instead, it encounters a flatter circular curve that gradually redirects the wheel.

European tangential geometry switch point rails are usually manufactured from special rolled rail sections that are not symmetrical about their vertical axes. These asymmetrical switch point rail sections are also shorter in height than switch stock rails, thereby permitting the switch slide plate to bear down on the base of the stock rail so as to resist rollover. The CEN 60E1A1 (formerly Zu1-60) section (see **Figure 6.5.1**) is a typical asymmetrical point rail section that has been used with 115 RE tee rail. (Some manufacturers prefer to instead use the somewhat shorter 54E1A1 section with 115 RE as it permits the risers to be somewhat more robust.) The difference in rail cross sectional configuration and height requires a shop-forged connection between the asymmetrical switch point rail and the common tee rail used in the turnout closure curve. Accordingly, these tangential design switches employ a floating heel design by default.

Tangential geometry switches have an extended zone within the switch point zone where the top of the switch point rail is very thin. This zone can experience accelerated wear, and the point rail may require buildup welding repairs or replacement far earlier than an AREMA-design switch point rail. Also, the lead distance for a tangential geometry turnout is typically much longer than for an ordinary secant switch turnout with the same frog number; hence, tangential geometry turnouts may not fit in areas of constrained track geometry.

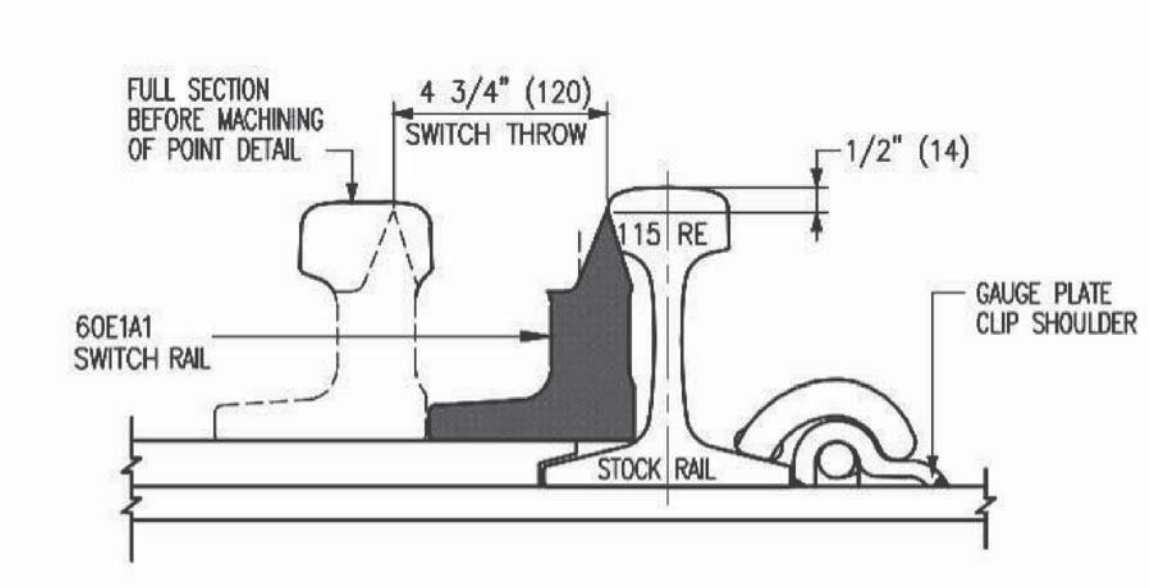


Figure 6.5.1 60E1A1 (formerly Zu1-60) rail section for a switch point

A few North American manufacturers are now producing proprietary tangential geometry switch point rail designs. These may be appropriate for some applications on a light rail transit system but are not generally warranted. As a guideline, tangential geometry turnouts should be considered whenever high speeds or a large number of movements must be made through the diverging side of a turnout.

6.5.4 Switches for Embedded Track

Turnouts in embedded track are a signature characteristic of light rail transit systems. Whenever rail transit track must be paved or embedded to permit either rubber-tired vehicles or pedestrians to travel along or across the track area, conventional ballasted track split switches—either conventional or tangential design—are generally impractical. The switch point "throw," the distance the switch point rail needs to move from one orientation to another, results in an unacceptably large void in the pavement surface. This void is dangerous to roadway vehicles and pedestrians. Voids also tend to collect debris and dirt, which impair switch operations. To deal with these difficulties, trackwork designers long ago developed what are known as "tongue switches."

A tongue switch consists of a housing that incorporates the three rails that converge at any switch. The switch rail (typically called the "switch tongue" or simply "tongue") is usually located in a roughly triangular opening in the center of the housing. The switch tongue is often grooved on its top surface to create a flangeway and either pivots or flexes on its heel end. This movement directs the wheel flange to either the straight track or the diverging track.

Tongue switches can be used in pairs (a "double tongue" switch), or a single tongue switch can be paired with a "mate." A mate is a rigid assembly, somewhat similar to a frog, which is typically placed on the outside of the switch curve. A mate has no moving parts; it has two intersecting flangeways. The mate ordinarily does not steer the wheels; it only provides a path for the wheel flange. All guidance must therefore come from the companion tongue switch. Traditional North

American street railway operations used tongue switches and mates almost exclusively until very recently. Because the mate provides no wheel guidance, it is generally considered a poor choice for transit lines that contemplate operating low-floor LRVs that have independently rotating wheels.

If, as is common, the mate is a flange-bearing design and the LRVs have solid axles, the larger rolling radius of the outside wheel on the mate compared to the inside wheel on the inside tongue switch, will steer the truck in the direction of the turnout curve. This obviously does not apply to trucks with independently rotating wheels (IRWs). Note also that this steering action would occur regardless of whether the LRV is negotiating the straight or the curved side of the switch. In a street environment, tongue switches and mates are much easier to keep clean than the conventional tee rail split switches. The mate component, having no moving parts, is especially well suited to a street environment because the flangeways are no deeper than those in the adjoining track and are thus relatively easy to keep clean.

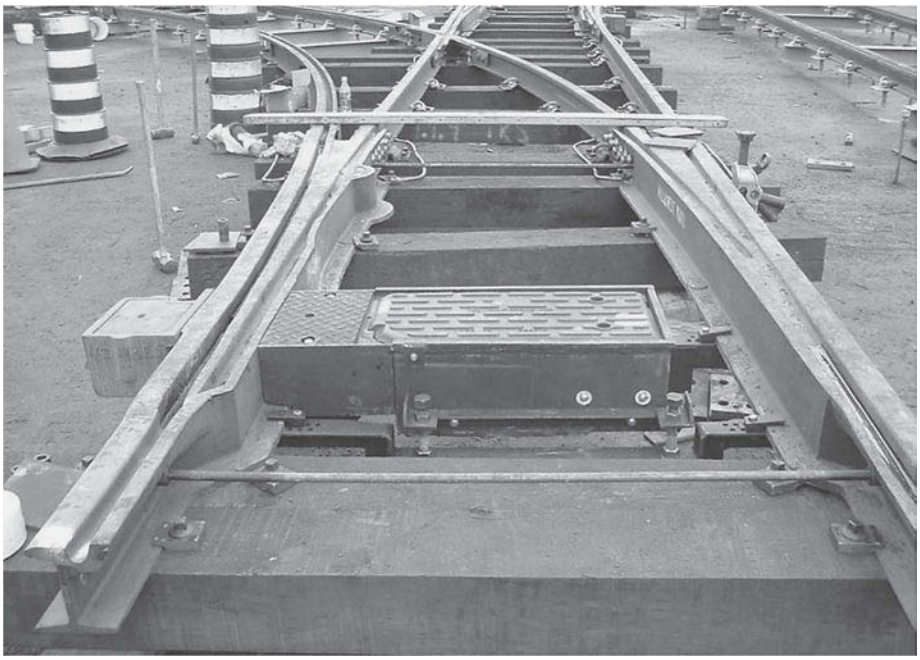
6.5.4.1 North American Tongue Switch Designs

North American tongue switches are typically constructed of solid manganese steel and are generally designed as illustrated in the 980 series of drawings in the AREMA *Portfolio of Trackwork Plans*. Those drawings show double tongue switches and a tongue switch/mate design. While these examples are conveniently available, a detailed examination is required to appreciate the differences between the AREMA designs and the configurations used by traditional street railway operations. **Figure 6.5.2** illustrates a typical tongue switch and mate turnout designed in accordance with the practices of the former American Transit Engineering Association (ATEA).[6]

Characteristics of the ATEA tongue switch design include the following:

- Traditional street railways (transit systems) in North America typically employed tongue switches and mates rather than double tongue switches, which were more common for railroad service. This was probably due to a desire to reduce the number of moving parts to be maintained, a key factor on large streetcar systems that could have hundreds of switches in embedded track.
- Tongue switch and mate designs for street railway service, as well as modern flexible double tongue switches, are typically curved throughout their length, with the point of the tongue recessed into the switch housing so that both the gauge face and the guard face of the tongue provide a smooth transition to the guarded rail immediately ahead of the tongue. The nearly tangential geometry results in turnout lead distances that are much shorter than those for straight tongue switches. Tongue switches with radii as short as about 50 feet [15 meters] were not uncommon.
- The flangeway widths in traditional street railway tongue switches and mates were narrower than those for railroad service. Track gauge was also usually unchanged from tangent track. The AREMA designs, on the other hand, have extremely wide flangeways and widened track gauge to accommodate steam locomotives with longer trucks, multiple axles, and large-diameter driving wheels. These factors make railroad tongue switch designs ill-suited for light rail vehicles that have shorter trucks, dual axles, narrower wheel treads, and almost

always appreciably smaller wheel diameters. The wide flangeways in the trackwork are also hazardous to pedestrians.



(Photo courtesy of Irwin Transportation Products)

Figure 6.5.2 ATEA tongue switch and mate turnout (shop assembly)

Typically, the switch tongue is placed on the inside rail leading to the diverging curve, so that truck steering action is provided by the interaction between the back side of the wheel flange and the tongue. This produces reliable steering of the truck due to the curved tongue providing a continuous guard. Some tongue switch designs amplify this guarding by depressing the wheel tread level of the diverging movement immediately beyond the point of switch, as shown in Section B of **Figure 6.5.3**. This causes the tongue to become an even more effective guard because it is higher than the wheel tread.

Switch tongues and, more importantly, the switch housing cavity require frequent maintenance to keep them clean and tight. Traffic riding on top of a rigid tongue tends to loosen and rattle it. For that reason, many properties positioned tongue switches on the outside of the curve for turnouts that were used either infrequently or only for converging movements. With the tongue positioned on the outside of the curve and the mate on the inside, straight through LRV wheel movements do not ride on the tongue, providing a quieter street environment. Note, however, that with the mate on the inside of the curve, outside tongue switch turnouts are not fully guarded. The deletion of a continuous guard through the critical switch area can result in derailments under some circumstances. Accordingly, outside tongue switches were typically not employed on switches with radii of less than about 100 feet [30 meters].

The old AREMA switch tongue design pivots on an integral cylinder that is positioned beneath the heel of the tongue. This cylinder is held in place by wedges on either side that are tightened by large-diameter bolts. These wedges tend to work loose as both they and the cylinder wear, causing the tongue to rattle and rock, which leads to noise and accelerated wear. Tightening the

wedges will only temporarily correct the problem, and over-tightening can make the switch difficult to throw.

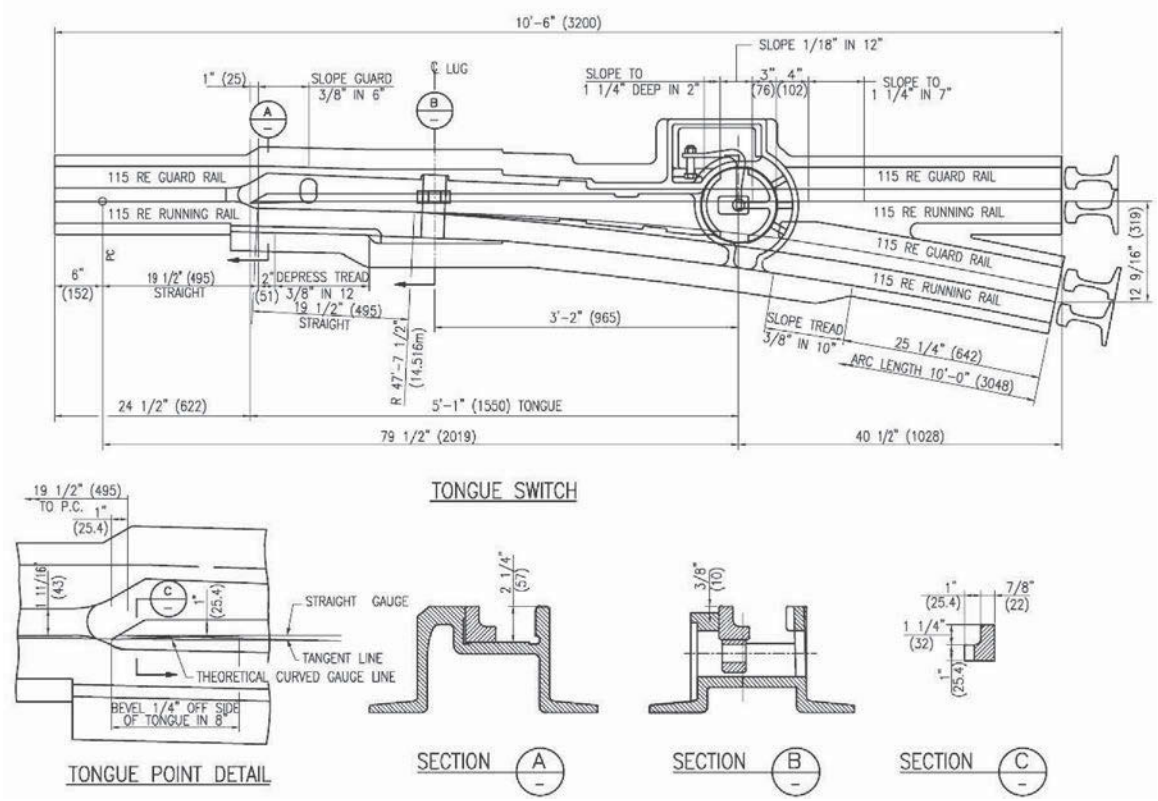


Figure 6.5.3 ATEA 75' radius solid manganese tongue switch

The ATEA standard tongue switch included a pivoting heel design that could be locked down by lever action. American special trackwork fabricators produced several other proprietary heel designs. These alternative heel designs generally required less maintenance and performed better in street railway use than the AREMA designs, but may have been ill-suited to the heavy axle load demands of railroad service. Manufacturers of these alternative designs are no longer in the transit industry, and the patents on their designs have very likely lapsed, placing them in the public arena.

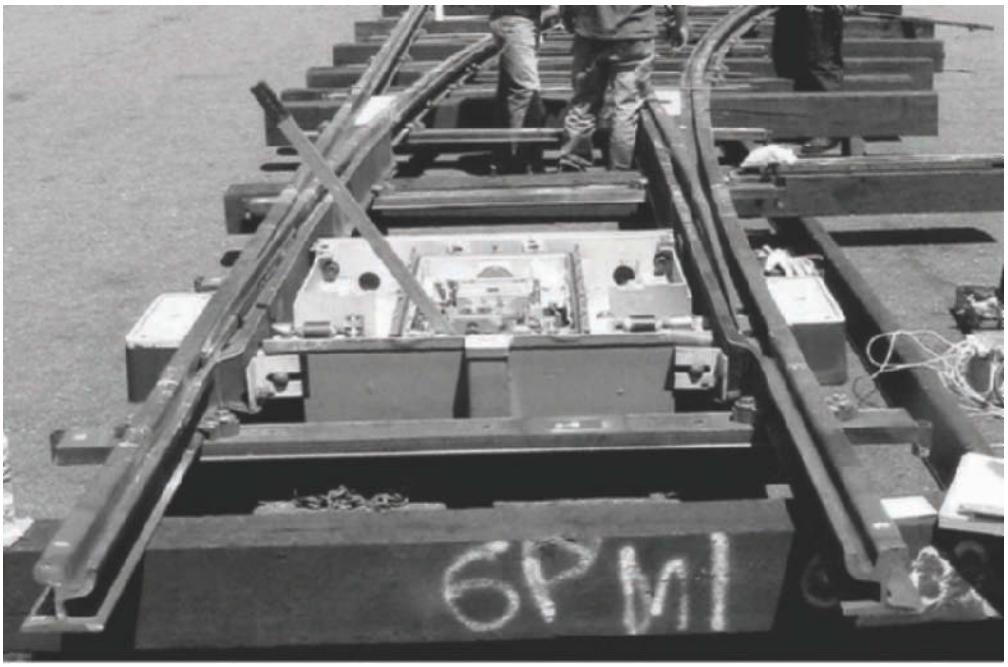
Standard American designs of tongue switches and mates were typically fabricated as manganese steel castings, similar to solid manganese steel frogs. Some alternative designs were partially fabricated from either rolled steel girder or tee rail sections. Tongue switches and mates have always been expensive items because it is difficult to produce large castings to precise tolerances.

Transit experience suggests that low-floor LRVs that incorporate independently rotating wheels (IRWs) do not have reliable steering through tongue and mate turnouts and that double tongue turnouts are preferred. The researchers have not been able to confirm that this is actually the case. Notably, the specifications for Toronto's pending (as of 2010) purchase of new streetcars for their legacy system stipulate that they must work reliably with single tongue switches. How closely the manufacturer will meet that goal is unknown. Prudence suggests that any new LRT

system that will be using LRVs with IRWs or might use such articulated car designs in the foreseeable future, should likely specify double tongue switches. However, that choice has significant capital, operations, and maintenance cost ramifications. The success or failure of the Toronto vehicle procurement is thus of great interest.

6.5.4.2 Double Tongue Flexive Embedded Switches

European light rail manufacturers developed flexible tongue switches in the post-WWII era, and these are virtually universal on European tramways and in-street LRT operations. A typical flexive double tongue switch is illustrated in **Figure 6.5.4**. The only locations where European LRT systems use a rigid mate in lieu of a second switch tongue is in complex layouts where overlapping turnouts make it impossible to provide the second tongue. In nearly all cases the tongues are rigidly fastened at their heel and flex, rather than pivot as is the case with North American designs.



(Photo courtesy of VAE Nortrak)

Figure 6.5.4 Flexive double tongue switch

European design flexive tongue switches are typically fabricated from special sections of rolled rails (CEN “construction rail” sections) and flat steel plate sections that are machined and arc welded to produce the switch fabrications. These fabricated designs are considerably less expensive to manufacture than the solid manganese steel castings typically used in North American tongue switches and mates but also may be less robust in service. Perhaps in response to that issue, some special trackwork fabricators now produce flexive double tongue switches in cast manganese steel that can then be hardened by an explosive hardening process. The tongue switches shown in Figure 6.5.4 are cast manganese steel.

Many North American light rail operators, both legacy systems and newer LRT operations have procured flexive double tongue switches. In-track performance of these installations has varied. Legacy street railway operations, in particular, often rate fabricated flexible tongue switches as

inferior to the robust design of the cast manganese steel tongue switches and mates, particularly with respect to wear. The number of issues with these designs has diminished with increased experience and the incorporation of better materials. Special surface hardening treatments can be incorporated into the design of flexible tongue switches to provide enhanced protection against wear. Refer to Chapter 5, Article 5.2.5.

6.5.4.3 AREMA-Style Split Switches in Embedded Track

Several LRT projects have elected to take ordinary AREMA-style split switches of open track design and use them in embedded track. **Figure 6.5.5** illustrates a typical installation. The switch assembly has been enclosed in a steel box that is equipped with bolted down but removable covers. The rail fastening systems within the boxes are functionally identical to the switch plates used in an ordinary open track installation, and the anchorage of the boxes to an underlying track slab effectively takes the place of the switch ties in an open track installation. The switch machine is mounted beneath the covers in the center of the track. The entire installation has then been insulated from the surrounding pavement, typically in a "bathtub" configuration.

There are two major issues associated with designs such as the one shown in Figure 6.5.5; both are associated with the large number of wide openings around the switch components:

- It is virtually impossible to exclude water, debris, snow, and ice from entering the switch boxes and affecting the operation of the switch. These units require an extensive amount of maintenance attention just for cleaning, and each such session requires removal and reinstallation of the cover plates.
- The wide openings are inconsistent with ordinary pedestrian traffic and are even more difficult negotiate for those with mobility impairments. Even though pedestrians should be excluded from the vicinity of any type of embedded switch, it is inevitable that they will occasionally be there. There is no way that a switch of this type can comply with ADAAG.

Some projects that have embedded split switches of this type have gone to great effort to reduce the switch throw to a small dimension, seemingly in an effort to comply with ADAAG, but have done nothing about the much larger openings elsewhere around the switch. In addition, reducing the switch throw to a small dimension can actually create a flangeway pinch point on the back side of the switch rail where the side planing ends. Repeated impacts to that location by the back face of the LRV wheel are transmitted throughout the entire switch assembly. One LRT operator had a major derailment when the repeated wheel impacts to the back of the switch rail finally loosened the connections between the switch rails and the switch machine, resulting in the switch point being gapped open. It is therefore strongly recommended that reducing the throw in any AREMA-design split switch, embedded or not, below the dimensions recommended by AREMA not be done without close scrutiny of the resulting flangeway clearances.

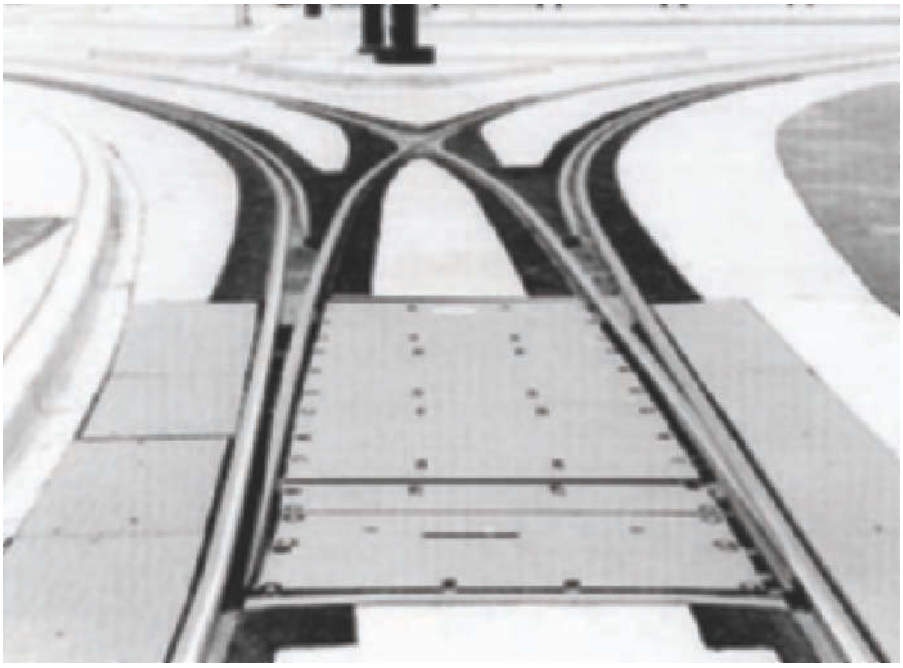


Figure 6.5.5 Embedded tee rail switch

6.5.4.4 Design Guidelines for Embedded Switches

If pedestrians cannot be reliably excluded from the vicinity of an embedded turnout-—which is usually the case—embedded switches should use either traditional North American street railway tongue switches and mates or European design flexive double tongue switches. AREMA tongue switch and mate and double tongue switch designs should not be used, as the flangeway openings are too large for areas where the general public has access.

Embedded switch flangeway design must conform with the vehicle wheel gauge and specifically to wheel flange widths. The design must consider flangeway openings and clearances in both switch tongue positions. If the embedded switch machine is designed for transit type switch point rail movements (typically a 2- ½-inch [64-mm] throw) the corresponding wheel gauge must also be within transit limits.

6.5.4.5 Switch Tongue Operation and Control

The switch throw of a tongue switch must be extremely short to preserve the switch tongue's ability to perform as an effective guard and to keep the open point flangeway as narrow as possible. The ATEA switch throw distance was only 2.5 inches (64 millimeters); a steel company designed an even shorter throw of 2.25 inches (57 millimeters). Such small switch throws are completely outside of the adjustment range of any standard railroad power switch machine of North American design.

Instead, traditional North American street railway properties employed switch machines that were essentially a large solenoid. Depending on the current flow direction in the solenoid field, the switch would either be thrown to the opposite orientation or remain in its present position. Once thrown, the tongue was held in place by a spring-loaded toggle. The toggle kept the tongue in place until the solenoid was activated to throw the switch in the opposite direction. It also made

the switch trailable without having to first throw the switch. The most common design, which is still in production, was known as a "Cheatham" switch, after its original manufacturer. A major drawback of the solenoid design is that the spring toggle neither locks the switch tongue(s) in place nor confirms that the tongue has actually thrown completely. This makes it possible for a switch tongue to accidentally throw under a rail car. Some North American operators equipped Cheatham switches with point detection relays that verify electronically that the switch tongue has been completely thrown.

European suppliers developed more modern switch machines for tongue switches that provide a greater throw force, detector rods, and a modified form of internal point locking. However, their design philosophy does not fully comply with conventional North American signal practice such as the AREMA Signal Manual. In addition, some transit properties have found it very difficult to maintain these machines. The current manufacturer of the Cheatham switch has made significant changes to its design so as to incorporate point detection and some degree of point locking. This situation is fluid, and readers are encouraged to confer with vendors for current product information. See Chapter 10 for additional discussion on switch machines.

6.5.4.6 Embedded Switch Drainage

Embedded switch and enclosure housings, regardless of design, create an opening in the street surface that will inevitably fill with storm water runoff and miscellaneous debris that is blown or washed into the openings. A positive drainage system of adequate capacity must be provided. It should permit solid debris to be flushed away. The switch design should promote free drainage at each end of the housing (as may be necessary to compensate for direction of track grade) or any cavity. A tongue switch installation could easily have half a dozen discrete separate locations where storm water could accumulate and hence must be drained. The design should also allow access into all cavities to enable cleaning out any solid material that may accumulate. Leaving such debris in place can interfere with the operation of the switch, promote corrosion, and facilitate stray currents. If the design includes cavities that are not essential to operation of the switch, but are likely to cause problems if they become filled with water or debris, the designers should consider filling such areas with a non-conductive material, such as an epoxy grout, prior to installation in track. The maintenance program should include sweeping, vacuuming, flushing, or blowing out embedded switch housings on a regular schedule plus as-needed cleaning after a major storm event, as well as routine scheduled inspections to verify that the drainage systems are clear and functional.

Some LRT systems have had a related problem from an unlikely source—municipal street sweepers that unintentionally sweep debris into the embedded switches, compounding maintenance issues.

Corrosion of threaded fastenings in embedded switches can make them impossible to adjust. All threaded fastenings in embedded switches should be made of corrosion-resistant materials, such as bronze or stainless steel, to avoid such problems.

6.5.4.7 Embedded Switch Heaters

Switch cavity drainage is especially important in northern climates that must contend with snow and freezing conditions. Even a well-designed switch drainage system can be foiled by freezing slush and runoff that accumulates in switch cavities, interfering with switch machine operations

and the closure of switch tongues or point rails. LRT systems located in places where snow and ice are even occasionally a problem would be well advised to incorporate heaters into each switch. These could assume various forms, but embedded switch heating systems must be provided with access points that permit inspection and, if necessary, replacement of the heating element without extracting the entire switch housing from the embedding pavement. The switch heater system must be configured so that insulating elements, such as the elastomeric grout commonly used, are not damaged or compromised by overheating.

6.5.5 Fully Guarded Tee Rail Switch Designs—Ballasted Track

The preponderance of special trackwork derailments occur at switches. Providing a guard in the switch area can be very beneficial, particularly if the turnout curve immediately beyond the switch is sharp and protected with a restraining rail. Readers may have noted that embedded tongue switch and mate and double tongue turnouts can provide a continuous restraining rail through the entire turnout. This includes the critical switch area, where the vehicle trucks must first make a change of direction. Rail transit systems that have extremely sharp turnouts in open track often employ what are variously known as either “house top” or “cover guard” switches. These switch designs are the signature component of “fully guarded” turnouts. A typical house top double-point switch is illustrated in **Figures 6.5.6** and **Figure 6.5.7**. As the name implies, a fully guarded turnout is one in which the diverging movement through the turnout includes continuous guarding from ahead of the point of switch to beyond the zone normally occupied by the frog guard rail.

The switch area provides most of the unique characteristics of a fully guarded turnout, including the following:

- The house top guard piece, which is positioned above the straight switch point, protects the critical first 12 to 18 inches [300 to 450 millimeters] of the diverging switch point by pulling the wheel set away from it. Because the house top is rigidly fixed and must allow the passage of a wheel that is traveling on the straight switch rail, it does not provide any guarding action for lateral moves beyond the immediate vicinity of the point of the switch. The house top is usually a continuation of a conventionally designed restraining rail that is placed in the tangent track ahead of the switch point. In addition to providing guidance for a facing point movement, this extended guard will protect the straight stock rail from wear during trailing point movements.
- The straight switch rail is a “double point” and provides a continuation of the restraining rail along the curved stock rail from the house top through to and including the heel of the switch.

Note that the spread at the heel of the switch is much larger than in conventional AREMA split switch design. This facilitates a robust connection between the double-point switch and the restraining rail.

In order for the double point to act as an effective restraining rail, the switch throw must be as short as possible. A throw distance no greater than 3 ½ inches [89 mm] is required, and a shorter throw dimension would be preferred. The normal throw distance for a powered switch in accordance with standard North American railroad practice is approximately 4 ¾ inches [121 mm]. Most conventional North American power switch machine designs allow for an adjustment

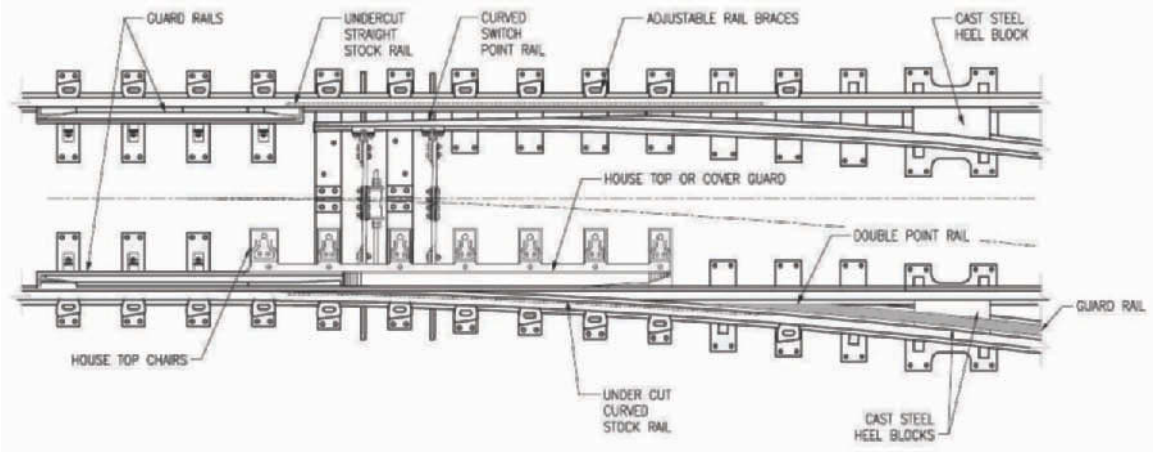


Figure 6.5.6 Fully guarded house top switch

Figure 6.5.7 Fully guarded switch with house top and double point

of 3 ½ to 5 ½ inches [89 to 140 millimeters]. If the machines were set to the smaller dimension, they would have no adjustment left for wear. Hence, a power switch machine for a house top switch must be custom designed. North American signal equipment manufacturers can provide machines with short throws; however, the locking rod design cannot be as robust as those provided with ordinary switch machines. In addition, since the switch rail is acting as a restraining rail, lateral forces applied to it by the wheels are transferred via the switch rods to the switch machines. This makes switch machines and connecting rods high-maintenance items that can require frequent adjustment.

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Chapter 7—Structures and Bridges

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CHAPTER 7—STRUCTURES AND BRIDGES

7.1 INTRODUCTION

This chapter principally discusses the interaction between railway tracks and aerial structures, such as bridges and viaducts, that carry them and presents the items to be considered during the design of aerial structures. Additional discussion is provided concerning the design of slabs-on-grade for supporting embedded and direct fixation trackforms.

Railway aerial structures come in many forms, and each has a different level of interaction with the tracks carried. At one extreme are heavy ballasted track bridges that have very little structural interaction between the rails and the structure. At the other end of the spectrum are concrete deck structures with continuous welded rail (CWR) directly affixed to the deck. This design is very typical of modern rail transit aerial structures. These structures can have significant interaction between the rail, which does not move, and the structure, which must expand and contract with changes in temperature. Somewhere in the middle of the spectrum are open deck bridges, which are very commonly used for long railroad structures and are often found on older urban rapid transit railways. These lighter structures generally use jointed rail to limit the interaction between the rail and the structure, but many have been successfully upgraded to use CWR. Finally, very nearly in a class by themselves, are bridges that have rails embedded into a concrete pavement, thereby allowing the trackway to be used by both rail and rubber-tired vehicles.

The design of aerial structures for light rail transit systems involves choosing a design code, determining light rail vehicle (LRV) forces, confirming track configuration requirements, and applying rail/structure interaction forces. This interaction is affected by such factors as the bearing arrangement at the substructure units, trackwork terminating on the aerial structure, type of deck construction, and type of rail fasteners. The details of the trackwork design significantly affect the magnitude of the forces that must be resisted by the aerial structure. In order to efficiently design an aerial structure for an LRT project, it is critical for the structural engineer to coordinate early and continuously with the trackwork engineer to fully understand the trackwork-related issues that affect the design of an aerial structure. Structural engineers should be involved in the project as early as the planning phase to provide support to the trackwork engineer.

Although they are not an aerial structure component, slabs-on-grade present challenges in structural design and detailing on current-day LRT projects. This chapter discusses some of the issues that warrant consideration during the design phase of slabs-on-grade.

7.2 DESIGN CODES

As of 2010, there is no nationally accepted design code that has been developed specifically for light rail transit aerial structures. In addition to owner-specific and local design codes, designers must choose between the *Standard Specifications for Highway Bridges,* published by the American Association of State Highway and Transportation Officials (AASHTO), the *Manual for Railway Engineering* (MRE) issued by the American Railway Engineering and Maintenance of Way Association (AREMA), and, more recently, the AASHTO *Load and Resistance Factor*

Design (LRFD) *Bridge Design Specifications*. Unfortunately, neither set of AASHTO specifications nor the AREMA manual accurately defines the requirements of an aerial structure to resist light rail transit loads, although the AASHTO specifications are generally more applicable.

It has been common for transit agencies to base the aerial structure design criteria on their state department of transportation's (DOT's) highway bridge design criteria. Prior to 2007, those criteria were typically derived from the AASHTO *Standard Specifications for Highway Bridges*. In October of 2007, the Federal Highway Administration (FHWA) mandated that new and replacement highway bridges that are part of federal-aid funded projects are to be designed according to the AASHTO *Load and Resistance Factor Design* (LRFD) *Bridge Design Specifications*. As such, if transit agencies continue electing to follow local DOT highway bridge design criteria, the AASHTO LRFD criteria will apply. Research shows that several recent LRT projects' aerial structure design criteria have been established as LRFD based. This is evidence of LRT aerial structures moving toward being designed according to the LRFD methodology, and this apparent trend will likely continue into the future.

Most light rail transit loads are greater than the HS20 truck load used by the AASHTO specifications, but light rail transit loads are much less than the Cooper E80 railroad loading cited in the AREMA manual. **Figure 7.2.1** plots bending moment versus span length for the Cooper E80 railroad load, the HS20 truck load, and the LRV load from the Dallas and St. Louis transit systems. As shown in Figure 7.2.1, for a 100-foot [30.5-meter] span, the LRV produces a bending moment approximately 50% higher than that produced by the HS20 truck load, but less than 20% of the bending moment caused by the Cooper E80 railroad load.

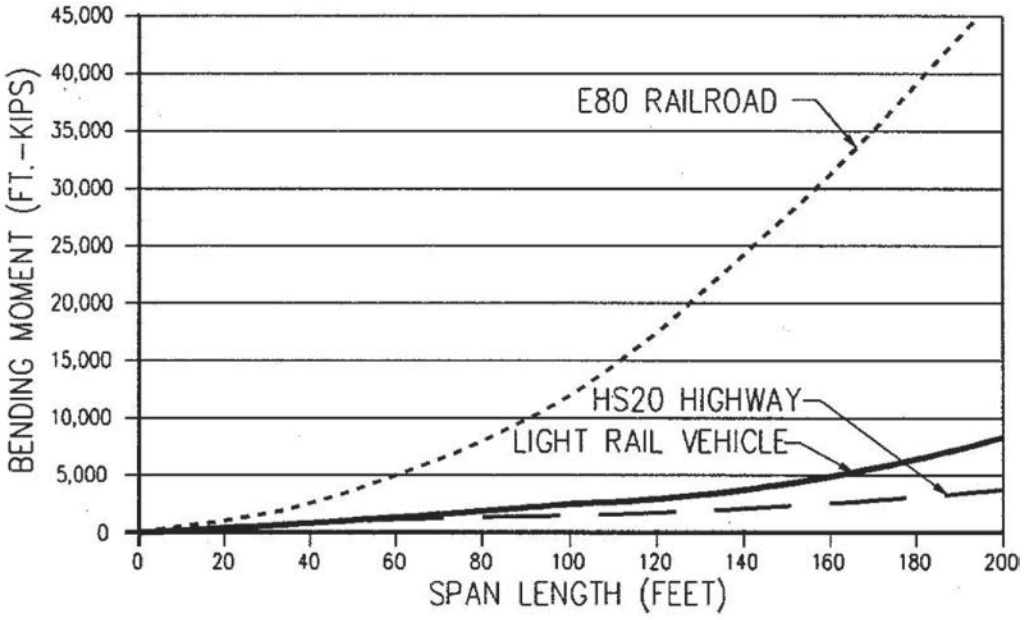


Figure 7.2.1 Vehicle bending moments on simple spans

The LRFD-based live load (designated as HL-93) has not been plotted in Figure 7.2.1 as it is a "notional" load and not intended to specifically represent a group of axles or a specific truck configuration. Also, the HS20 live load moments cannot be directly compared to the HL-93 moments since a unique set of load factors applies to each according to the AASHTO Standard Specifications (for HS20) and the AASHTO LRFD Bridge Design Specifications (for HL-93), respectively.

The AREMA *Manual for Railway Engineering*, although applicable to railroad structures, is too restrictive for light rail transit structures due to the great difference in loadings. Wheel spacings for AREMA loading do not correspond to those found on LRVs, and the AREMA impact criterion is not consistent with the suspension and drive systems used on LRVs. The service conditions, frequencies, and types of loading applicable to freight railroad bridges are not consistent with those items on dedicated light rail transit systems.[1], [2]

A strong similarity exists between light rail transit design requirements and the AASHTO specifications. For light rail transit aerial structures, the ratio of live load to dead load more closely approximates that of highway loadings than that of freight railroad loadings. In addition, since the magnitude of the transit live load can be more accurately predicted, the conservatism inherent in the AREMA *Manual for Railway Engineering* is not required in light rail transit structures.

It is interesting to note that the older "legacy" rail transit systems (e.g., in Chicago, Philadelphia, and New York) often refer to the AREMA *Manual for Railway Engineering* for the design of bridges, but the newer systems (e.g., in Atlanta, Washington, D.C., and Baltimore) base their designs on the AASHTO specifications. This is partly due to an increased understanding of an aerial structure's behavior and the designer's confidence in the ability to accurately predict transit loads. Although there is no current bridge design code that is completely applicable to light rail transit bridges, the use of the AASHTO specifications will result in a conservative design that is not overly restrictive or uneconomical.[1], [2], [3]

7.3 VEHICLE FORCES

The vehicle forces applied to an aerial structure are often set by the transit agency's design criteria for site-specific circumstances. In addition to the loadings imposed by trains of rail vehicles, many rail transit properties include other, heavier vehicles in their design criteria for aerial structures. These vehicles might include a crane car, an overhead lines maintenance car, a work train consisting of ballast cars and flat cars pulled by a locomotive, and even highway vehicles (during construction). On the other hand, some transit properties establish the LRV as the sole basis of design for the aerial structures.

In addition to the LRV and alternative vehicle live loads applied to the aerial structure, the following vehicle-related forces are also typically considered in structural design:

- **Vertical impact.** This represents a dynamic load allowance that is applied to the static wheel load to account for wheel load impact from moving vehicles.
- **Horizontal impact.** Similar in nature to vertical impact, this vehicle force is associated with dynamic effects but is applied horizontally to the aerial structure.

- **Centrifugal force**. This accounts for the radial force and overturning effect resulting from a vehicle traveling through a horizontally curved alignment.
- **Rolling force (vertical force applied at each rail, one up and one down).** Also known as the rocking effect, this vehicle force accounts for the inherent rocking of the vehicle back and forth during its travel.
- **Longitudinal force**. This results from braking and acceleration/tractive effort.
- **Derailment forces.** These can be considered analogous to collision-type loadings or other extreme event loadings associated with highway bridge design. Derailment forces are attributed to LRV(s) coming off of the running rails, which results in a lateral excursion of the vehicle from the centerline of track. Typically, derailment loads are applied both vertically and horizontally to the structure and affect the superstructure and substructure designs. Derailment forces must be developed in conjunction with the trackwork configuration used on the aerial structure. Given the common use of plinths, guard rails, and/or restraining rails, the physical possibility of any significant lateral excursion of the vehicle should be considered early in the design phase. Guard rails and restraining rails limit lateral excursion and, as a result, minimize some force effects due to vehicle derailment.

Combinations of vehicle forces, in conjunction with dead loads, wind loads, thermal loads, and seismic loads, are developed to generate the load cases that govern the design of an aerial structure.

7.4 STRUCTURE AND TRACK CONFIGURATIONS

7.4.1 Structure Type Considerations

During the early stages of design, the designer must determine the type of superstructure and substructure to be used for a specific aerial transit structure. Whether the superstructure is composed of steel or concrete girders, as well as the configuration of the girders, must be evaluated with respect to the project and site constraints.

Commonly considered superstructure types include the following:

- Cast-in-place concrete
- Precast concrete girders with cast-in-place concrete deck slab
- Segmental precast concrete
- Steel girders with cast-in-place or precast concrete deck slab
- Steel box section with either cast-in-place or precast concrete deck

The following factors are used to comparatively study superstructure types:[1], [4], [15]

- Effectiveness of structural function (span lengths, vertical clearances, span-to-depth ratio, etc.)
- Constructibility issues, such as erection and construction convenience, including transportation of the structural elements to the site
- Production schedule constraints

- Capital cost
- Maintenance cost
- Availability of materials and finished product
- Availability of construction expertise
- Site working conditions, including weather, local ordinances, and working restrictions
- Aesthetics
- Owner's preference
- Urban constraints
- Durability
- Construction schedule

Substructure components typically are composed of reinforced concrete structures. Commonly considered substructure types include the following:

- Single-column hammerhead-type piers
- Multicolumn piers with rectangular caps
- Trestle bents
- Wall piers
- Abutments of various types

The substructure type and its supporting foundation play a role in the overall stiffness of the structure as related to the distribution of forces, as described in Article 7.5.4. Substructure type comparisons must consider many of the same factors outlined above for making superstructure type selections.

In addition to the parameters listed in regard to choosing the optimum structure type, there are other design and performance issues that warrant consideration during the decision-making process. These include concerns with differential deflections related to station platforms adjacent to track structures, vibration issues, stray current isolation (corrosion control), and miscellaneous structural details.

- **Differential deflections.** In some cases, aerial station platforms positioned adjacent to the track structure may be required. This requirement depends on site constraints, geometric constraints, right-of-way limitations, and other project constraints. The Americans with Disabilities Act Accessibility Guidelines (ADAAG) provide limitations related to horizontal gaps between platforms and LRVs and limitations on vertical deflection between the top of the platform (walking surface) and the floor of the LRV. Detailed structural analyses are required to design the track structure and platform structure to satisfy the differential deflection criteria. Locating stations on-grade rather than on an aerial structure, if at all possible, is recommended.

- **Vibration.** The natural frequency of an aerial structure is directly related to the type and configuration of the superstructure. The span length, structure depth, and structural stiffness are variables that affect the structure's natural frequency. For instance, a long span steel structure with limited structure depth available may not be as stiff as a short span concrete girder bridge without structure depth limitations. Refer to Chapter 9 for further discussion associated with the vibration of the structure-track-vehicle system.

- **Stray current isolation.** The ability of the structural details to provide stray current isolation is an important consideration, and various details offer differing degrees of isolation. For example, steel bearings provide less stray current isolation than elastomeric bearings. In addition, locations of epoxy-coated reinforcement versus uncoated reinforcement can play a role in the overall stray current isolation plan. The weldability of the reinforcing steel is also a consideration as it affects the ability to achieve electrically continuous reinforcement in a concrete deck slab. Refer to Chapter 8 for further discussion related to stray current control issues.

- **Miscellaneous structural details.** Aerial structures not only need to support and accommodate the LRVs, but they also typically support utilities, safety handrailings, maintenance walkways, overhead contact system (OCS, also known as "catenary") poles, and other miscellaneous items. These items and their locations need to be carefully coordinated to ensure that required vehicle clearance envelopes are satisfied and that the functionality and maintenance needs of the various systems are accommodated. As an example, an OCS pole may need to be located between the two tracks on a double-track structure with direct fixation deck. The pole may be mounted to the substructure and would need to pass up through the deck. The structural details associated with an opening in the deck would need to be coordinated among the track, OCS, and structural engineers to ensure that the details accommodate the needs of each discipline.

Structure type selection will also consider the construction costs for each alternative. Historical cost data can be obtained from previously constructed LRT projects and extrapolated to current costs. In addition, most state DOTs archive cost history data related to highway bridge construction. These cost data are relatively easily obtained and can serve as an approximation of costs associated with the construction of LRT aerial structures. It is likely that these highway bridge cost items would need to be adjusted to apply to the specific aerial structure under design.

As part of the preliminary design effort for an aerial structure, a study should be performed to determine the most desirable structure configuration based on economic, social, environmental, and technical needs. Various span arrangements should be considered in the design. The final span length selection should include consideration of aesthetics and community factors in addition to structural efficiency.

Many times in an urban setting, span lengths are specified that provide the required horizontal and vertical clearances to existing facilities along the rail system's alignment. The location of existing railroad tracks, roadways, highway bridges, waterways, and major utilities can restrict substructure locations, thereby limiting the choices for span lengths.

7.4.2 Types of Deck Construction

There are four different types of bridge deck construction used in rail transit construction. Below, these four types of bridge deck construction are listed in roughly chronological order, based on when they were first used for rail transit guideways:

- The earliest elevated transit trackway (beginning around 1870) featured open deck construction, in which timber cross ties were attached directly to the steel superstructure. This type of construction is lightweight, especially compared to the ballasted deck construction described below, and is very common for railroad structures. However, radiated noise from open deck structures was a significant problem, particularly with relatively lightweight superstructures. Vibration of deep web girders on open deck elevated structures—a phenomenon sometimes called "oil canning"—amplified all other noises.

- Embedded deck construction became necessary very early for streetcar lines that crossed bridges that were shared with wagons, carriages, and other users of the public streets. In this kind of construction, the only portion of the track structure that is visible is the running surface on the tops of the rails. The remainder of the track structure is concealed by the roadway pavement.

- Ballasted deck construction, also common in railroad work, became common for urban rail transit around the beginning of the 20th century, partially in response to the public's complaints about the noise and vibration generated by open deck structures. However, ballasted deck elevated lines were very expensive due to the much heavier structures required. Track maintenance on ballasted deck aerial structures is also difficult, particularly the spot replacement of defective cross ties.

- Direct fixation deck was developed to resolve the shortcomings of the ballasted and open deck designs. The earliest direct fixation track sections (used in both subways and on aerial structures) consisted of timber ties and half-length ties embedded into a poured concrete deck. This type of construction was commonly employed for rail transit structures as late as the 1960s. In contemporary direct fixation track designs the embedded ties have been eliminated and, instead, rail fasteners are used that can be anchored into a reinforced concrete substrate and that incorporate rail restraint, acoustic attenuation, and electrical isolation features.

The articles below address each of these major structural trackforms. They are discussed in the sequence in which they most commonly appear on light rail transit projects.

7.4.2.1 Direct Fixation Deck Construction

Direct fixation deck construction is the standard practice for most transit properties for bridges that are 300 feet [90 meters] or longer. The current concepts, as developed in the 1960s for then-new heavy rail transit projects such as BART (San Francisco), WMATA (Washington, DC) and MARTA (Atlanta), have the rails attached to the concrete deck by resilient fasteners. The various possible methods of this attachment are detailed in Chapter 4, Article 4.6.

The advantages of the direct fixation trackform include the following:[2],[10]

- Absorbs noise and vibration and provides vertical flexibility with resilient direct fixation fasteners
- Improves aesthetics by using shallower, less massive structures
- Results in a relatively low dead load compared to other trackforms
- Provides both electrical isolation and a means to efficiently adjust the line and grade of the track with rail fasteners
- Requires less maintenance and is easier to maintain than alternatives
- Retains track geometry much longer than ballasted track
- Provides relatively good ride quality
- Offers relatively good live load distribution

While direct fixation trackwork is appreciably more expensive than ballasted track, the reduced dead load of the track means that the bridge superstructure and substructure can be less robust and hence less expensive, resulting in a net savings per unit length of route. The use of direct fixation track construction was credited with saving millions of dollars on one 1960s heavy rail transit project by eliminating the weight of cross ties and ballast.[14]

As detailed in Chapter 4, Article 4.6, direct fixation track can be configured in several different ways. That information will not be repeated here. The important thing to note for purposes of this chapter is that the practical construction tolerances for bridges, tunnel inverts, and slabs-on-grade are appreciably looser than those required for the top-of-rail profile of railway tracks. Installation of direct fixation trackwork therefore requires either unusually tight construction tolerances for the underlying support structure or a relatively simple way of compensating for deviations. In the case of bridges, the construction tolerances for the construction of the deck, including residual deck camber, are usually far less stringent than are necessary to provide a satisfactory top-of-rail profile.

Because of this, most rail transit systems use a concrete pad, or plinth, approximately 6 inches [150 mm] tall to support the direct fixation fasteners and anchor them to the superstructure. Intermittent gaps are provided along the length of the plinths to accommodate deck drainage and to provide openings for electrical (systems) conduits placed on the deck. In addition, the deck slab sometimes incorporates recesses to accept the second-pour plinths. Each plinth recess forms a shear key to help resist the lateral loads from the rail and vehicles and also slightly reduces the effective height of the plinth. The second-pour concrete plinths are carefully constructed to meet the alignment and profile requirements of the CWR and fasteners.

Reinforcing steel dowels (typically in the form of inverted U-bars and often called “stirrups”) project from the bridge deck, anchoring the second-pour concrete plinths to the deck. (Per CRSI, “dowels” is the correct terminology for these bars, but “stirrups” is a common vernacular among transit trackwork designers and constructors.) The reinforcing in the concrete plinths should be designed to assist in resisting the loads imparted by the rail fastener anchors to the plinths. In addition to the compression forces on the plinths, the fastener anchors must resist horizontal

shear and tension forces as a result of the braking, accelerating, and lateral forces from the LRVs.

The reinforcing in the plinths needs to be designed in conjunction with the rebar dowels that project up from the concrete slab beneath the plinths. These dowels attach the plinth and fastener anchorage system to the supporting slab. Depending on the loads being resisted, the rebar dowels have been installed longitudinally along the length of the plinths or transverse to the plinths. In either orientation, the height that the dowels project above the slab depends on the details of the reinforcing in the plinths. Figures 4.6.1 through 4.6.5 in Chapter 4 of this Handbook illustrate common reinforcing steel details for concrete plinths. See Chapter 13, Article 13.3.2 for additional discussion on the relative merits of installing the dowels transverse to the track versus parallel to the rails.

Some transit projects have installed the dowels in the slab while the concrete was still wet, and other projects have drilled and grouted the dowels into the completed supporting slab. Should the drilling and grouting method be considered, the designer needs to evaluate the chance that the reinforcing in the slab will be damaged by the drilling, and the capacity of the grout is dependent on the proper mix being installed. With either installation method, it is critical that the contractor install the dowels at the proper height projecting above the supporting slab to properly engage the plinth reinforcing.

The typical light rail transit project includes a multitude of contractors and subcontractors working in the same corridor. As such, no matter which orientation of rebar dowels is chosen, the dowels need to be protected from damage by the follow-on construction activity. This is particularly important if the dowels are epoxy-coated rebars. One method to protect the dowels is to require the contractor to install temporary timber blocking that is the same height as the dowels and adjacent to them.

Figures 7.4.1 and 7.4.2 illustrate two stages of the construction of a bridge deck to support direct fixation track. In Figure 7.4.1, the deck is ready to be poured. The formwork has been positioned along the path of each rail so as to provide a recessed key in the deck for the direct fixation track plinths. The holes in the forms are for the placement of dowels when the deck is poured. In Figure 7.4.2, the deck has been poured and the formwork stripped, and the installation is ready for the construction of the direct fixation track system.

Although direct fixation is the current standard practice for deck construction, there are some disadvantages to consider. Disadvantages of direct fixation deck include the following:

- Rail/structure interaction must address thermal forces
- Relatively high initial cost
- Tight construction control required
- Specialized rail fasteners required

(Photo courtesy of Bryant Contracting, Inc.)

Figure 7.4.1 Direct fixation deck formwork for plinth dowels and recessed key for plinths

(Photo courtesy of Bryant Contracting, Inc.)

Figure 7.4.2 Completed deck with plinth recesses and dowels

7.4.2.2 Ballasted Deck Construction

Ballasted deck construction is still considered a valid choice by most transit agencies. It is usually used on short to moderate length bridges, generally 300 feet [91 meters] or less. Advantages of the ballasted deck include the following:[2],[4],[10]

- Provides an intermediate cushion between the rails and the structure to enhance ride quality.
- Limits the transfer of thermal forces from the track to the superstructure.
- Uses standard ballasted track rail fastening hardware.
- Reduces noise and vibration compared to open deck construction.
- Permits standard track maintenance methods to adjust alignment and profile.
- Provides good live load distribution and good track support.

Disadvantages of the ballasted deck include the following:

- The cost of deck waterproofing beneath the ballast layer.
- The relatively heavier deck load.
- The greater depth of deck required.
- Rail breaks can result in horizontal, vertical, and angular displacements.
- The cost of maintenance of the ballast layer, including track resurfacing (tamping) and periodic ballast cleaning. Without the latter, the ballast on a bridge will tend to lose much of its original resiliency.

7.4.2.3. Embedded Deck Construction

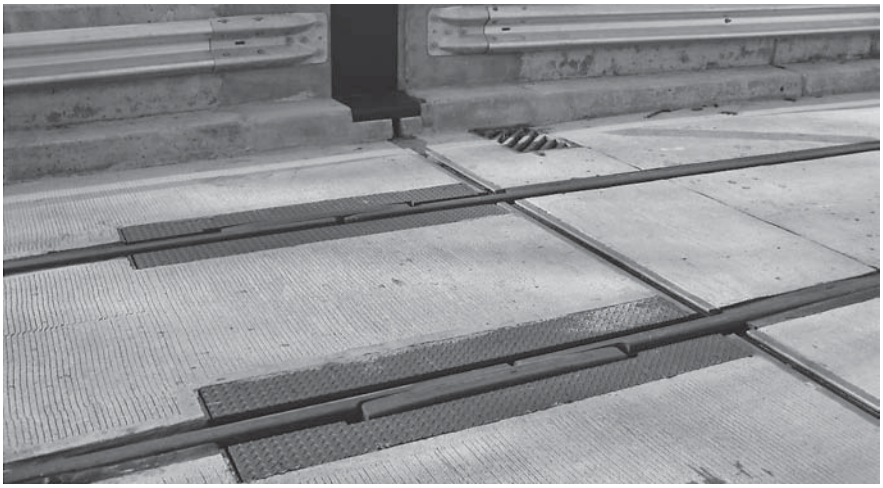
Where lanes on a bridge deck are shared by LRVs and rubber-tired traffic, the rails are embedded in the concrete deck so that the tops of the rails are at the same elevation as the top of the pavement.

Various items are considered when preparing the designs for the embedded rails, and the bridge engineer must coordinate extensively with the trackwork engineer at the early stages of the design process to understand the requirements of the track system. Some items for consideration are the following:

- Determine the dimensions and details of the trough or recess in the deck to accommodate the rails, fasteners, fastener anchorage items, waterproofing system, and stray current protection system.
- Check the strength of the concrete deck to support the LRV loads, especially in the thinner deck section beneath the rails. One method to avoid thickening the deck for the entire width of the bridge is to position beams beneath the deck directly under the rail locations.
- Carefully detail the reinforcing steel in the deck under and around the rail troughs in order to maintain the required deck strength.

- Determine the material that is to be placed adjacent to the rails to fill the trough up to the top of the deck. This material needs to be able to seal the trough so rain and snowmelt do not penetrate into the trough, compromising the service life of the deck surface. The fill material should be chosen to minimize the long-term maintenance of this area.
- Develop the transverse expansion joints in the deck that accommodate the bridge's thermal movement to efficiently interface with the rails passing through them. Special rail expansion joints may need to be detailed adjacent to these bridge expansion joints so that the thermal movement of the bridge is not restrained.
- Consider the rail/structure interaction forces that develop due to the restraint offered by the continuously welded rail in the rail troughs. Determine the gap in the rail that could develop should there be a rail break.
- Review the details of the deck drainage system in relationship to the embedded rails. The runoff crossing the deck will be interrupted by the rails and end up traveling longitudinally along the deck in the rail trough. A drainage system needs to be developed to periodically capture this runoff along the rails.
- With the loads from the LRVs being concentrated below the rail trough and not dispersed to a wider deck section by the plinths, the structural engineer has to carefully examine the superstructure to confirm that there are not any adverse effects. For steel structures, this includes an examination of the fatigue stresses and fatigue-sensitive connection details on the bridge.
- Review the surroundings of the aerial structure location to determine whether there are any noise-sensitive facilities beneath or adjacent to the aerial structure. The embedded track construction is likely to generate more noise from the LRVs running along the embedded track than the direct fixation alternative that uses plinths and fasteners on top of the deck. Without appropriate detailing, the entire bridge deck could resonate, amplifying the rolling noise that originates at the wheel/rail interface.

It is highly unlikely that any rail fastening design detail would permit the bridge superstructure to expand and contract independent of a rail that is embedded within it and do so reliably over the long term with minimal maintenance. Therefore, a rail expansion joint is virtually certain to be necessary at any expansion joint in the bridge deck. The rail expansion joint will necessarily provide an opening in the deck surface so that the rail ends can move relative to each other. That opening will inevitably become clogged with dirt and debris if the joint is not configured so that storm water will flush it to scuppers below the deck. **Figure 7.4.3** illustrates a rail expansion joint at the abutment of an embedded track bridge. Note that the rail expansion joint does not straddle the deck expansion joint but is instead fully supported by the abutment. See Chapter 5 for additional discussion of embedded rail expansion joints. See Chapter 8 for additional discussion of stray current issues.



(Photo courtesy of Trammco)

Figure 7.4.3 Rail expansion joint on embedded track bridge

7.4.2.4 Open Deck Construction

Although it is not used as commonly as either direct fixation or ballasted deck construction, open deck construction is a valid alternative for track support on bridges. Open deck construction is typically only employed when rehabilitating an older railroad bridge that, for whatever reason, cannot be changed to another trackform. (However, at least one LRT project has built entirely new open deck bridges.) **Figures 7.4.4 and 7.4.5** illustrate such structures. Notable is the non-conventional use of concrete AASHTO beams to support the ties rather than the usual steel girders or timber stringers.

Figure 7.4.4 Example of open deck viaduct LRT aerial structure

Figure 7.4.5 Detail of open deck on LRT aerial structure

As with the other types of bridge track construction, it is imperative that the bridge engineer coordinate closely with the trackwork engineer in order to design an effective open deck track support structure. Some items that need to be considered when evaluating the use of open deck construction include the following:

- The longitudinal spacing of the timber ties, based on track support requirements.
- The special details for the timber ties at skewed piers and abutments and at transitions to other types of track construction, such as direct fixation and at-grade track. Consideration needs to be given to the transition in the stiffness of the track support system in order to maintain a smooth ride for the patrons of the LRT system.
- The details for anchoring the ties to the superstructure and adjusting the ties to accommodate the superelevation of the track in curved track segments.
- The relative movements caused by the superstructure expanding and contracting relative to the fixed CWR.
- The material used for the ties where the track crosses over public elements, such as roadways and sidewalks, to prevent preservative treatment chemicals within and on the surfaces of the ties from dripping onto the areas below the track.
- Details of the track system to limit the noise generated by the LRV as it crosses over the open deck construction.
- Methods to maintain the track system, especially the gauge of the track, on an elevated structure.

- The maintenance and emergency walkway requirements and details.
- Coordination with emergency response teams so they are aware of the open deck configuration and provision of the means to evacuate people from the LRVs, if necessary, in a safe manner.
- Provision of sufficient lateral bracing in the superstructure framing. The lateral bracing system provides stability to the superstructure and resistance to potential superstructure overturning effects that can be associated with centrifugal forces in curved alignments and derailment forces.
- Confirmation that cross ties can be sufficiently dapped to accommodate the as-erected girder camber and deflection conditions.
- Evaluation of structural vibrations as related to rider comfort and dynamic interaction between the aerial structure and the LRVs.

7.4.2.5 Adding Rails to Existing Structures

There are instances when it may be advantageous and cost-effective to utilize an existing bridge for the construction of a light rail transit line. Although LRV loading is higher than the truck loads that highway bridges are designed to resist, an existing bridge may be a candidate for strengthening in order to add tracks for light rail transit.

Assuming that embedded track isn't required to accommodate mixed rail and highway traffic on the existing bridge, and no accommodation for future rails was provided when the bridge was originally built, concrete plinths would typically be added on top of the deck for anchorage of the direct fixation fasteners.

There are a number of items that need to be studied in order to confirm the feasibility of adding tracks to an existing bridge:

- A structural analysis of the existing bridge needs to be performed to determine the strengthening needed to support LRV loads. Even if the deck itself is structurally sound, it is unlikely that the rails will fall directly above existing stringers. So as to minimize shear stresses in the deck, supplemental stringers may be required below the deck directly beneath where the rails will be installed. The existing bearings need to be analyzed in addition to the superstructure and substructure. Careful consideration of the LRV loads needs to occur, since these loads are different in magnitude and direction than the loads from the highway trucks that the bridge was first designed to carry. For steel structures, the existing connection details need to be reviewed to determine whether any of them are fatigue-sensitive details, requiring an analysis of the fatigue stresses.
- The details of adding the concrete plinths to the bridge deck need to be determined. In order to drill holes in the deck to insert rebar dowels and grout to tie the plinths to the deck, it is highly desirable to identify the locations of the existing reinforcing in the deck so the drill holes do not damage the existing reinforcing. There is non-destructive test equipment that can be used to locate the existing reinforcing.

- In addition to the LRV loads, the existing bridge needs to be analyzed to address stray current corrosion, vibrations from the LRVs, and rail/structure interaction caused by the continuous welded rails restraining the thermal movement of the superstructure.
- The expansion joints in the deck may need to be retrofitted to accommodate the installation of the concrete plinths.
- The drainage patterns on the deck will be altered by the installation of the concrete plinths. Horizontal gaps should be included along the length of the plinths to permit runoff to flow to the existing deck drains. Alternatively, new deck drains can be added.
- The bridge will need to be retrofitted to provide support for the OCS poles, unless the bridge is short and the OCS poles can be installed at the approaches to the bridge. The barriers along the sides of the deck and the bridge piers are possible locations for the OCS poles. On a through truss bridge, it is typically possible to hang the OCS directly from existing cross members above the deck.
- As with embedded deck construction, the surroundings of the bridge location need to be reviewed to determine whether there are any noise-sensitive facilities beneath or adjacent to the bridge. Vibration of the deck and the webs of any tall steel girders can result in a noise phenomenon sometimes known as “oil canning.” Mitigation measures may need to be performed to address the rolling noise from LRVs crossing the deck.
- The details to transition the light rail tracks from the deck to approach areas beyond the abutments must be determined. Abrupt changes in track stiffness should be avoided.

7.5 RAIL/STRUCTURE INTERACTION

7.5.1 General

With widespread use of CWR, the designer of an aerial structure must be aware of trackwork design and installation procedures, as well as vehicle performance and ride comfort issues. Trackwork design and installation procedures are especially critical in establishing the magnitude of the interaction forces between the rail and aerial structure.

As the temperature changes, the superstructure (including both the deck and the supporting girders) expands or contracts. Depending on the type of trackform, the interaction of the track structure and the bridge structure will exhibit different behavior.

On an “open” trackform (ballasted, open deck, or direct fixation), rails are effectively stationary because of both their continuity throughout the length of the bridge and their being anchored off the bridge. In the case of ballasted track, the relative movement between the superstructure and the combined ties and rails is accommodated by slight movements of the cross ties within the ballast. Effectively, the ballast beneath the cross ties is a shear plane between the track and the bridge. In a direct fixation bridge, the movement of the superstructure relative to the rails as the temperature changes imposes deformation on the fastening system that attaches the rails to the bridge deck. Open deck structures are more indeterminate, and some relative movement can be expected between the rails and the rail fastenings and between the bridge ties and the girders.

7.5.2 Continuous Welded Rail

The majority of the early elevated rail transit systems used trackwork composed of jointed rail supported on simple-span guideway structures. Alternatives have been developed for modern rail transit trackwork on aerial structures. Rather than the classical jointed rail with bolted connections every 39 feet [12 meters], the trackwork is normally constructed with continuous welded rail. With either rail configuration, the rails can be fastened directly to the aerial structure's deck, installed on ties and ballast, or installed on ties without ballast.

The bolted connections used with jointed rail allow sufficient longitudinal expansion and contraction to reduce the accumulation of thermal stresses along the rails. But there are some disadvantages; bolted joints[4]

- Generate noise and vibration
- Are troublesome to maintain
- Contribute to derailments if not maintained
- Cause rail fatigue in the proximity of the rail joints
- Cause wear of the rolling stock
- Reduce ride quality
- Increase the dynamic impact forces applied to the aerial structure
- Are points of high electrical resistance for traction power return currents

CWR has been the most common track configuration for rail transit systems for several decades. This is mainly due to its ability to overcome many of the disadvantages of jointed rail. Specifically, CWR [5],[6]

- Minimizes noise and vibration
- Reduces track maintenance
- Improves track safety
- Eliminates the joints that cause rail fatigue
- Limits wear of the rolling stock
- Provides a smooth, quiet ride
- Limits the dynamic impact forces applied to the aerial structure
- Provides a consistent path for traction power return currents.

The use of CWR, combined with direct fixation of the rails to the supporting structure, is an improvement in the support and geometric stability of the trackwork. As a result, rider comfort and safety are enhanced, and track maintenance requirements are decreased.

The use of CWR requires designers of trackwork and aerial structures to consider issues that typically do not arise when using jointed rail, such as the following:[7],[8],[9]

- Providing sufficient rail restraint to prevent horizontal or vertical buckling of the rails
- Providing anchorage of the CWR to prevent excessive rail gaps from forming if the rail breaks at low temperature
- Determining the effect a rail break could have on an aerial structure
- Calculating the thermal forces applied to the aerial structure, the rail, and the fasteners as the aerial structure expands and contracts and the CWR remains in a fixed position
- Providing a connection between the CWR and aerial structure (direct fixation fasteners) that is sufficiently elastic to permit the structure to expand and contract without overstressing the fasteners

An important element in the design of trackwork using CWR is the consideration of rail breaks. Rail breaks often occur at structural expansion joints in the aerial structure and must be accommodated without catastrophic effects such as derailment of the vehicle. Depending on the length of the aerial structure, the CWR has to be sufficiently restrained on the aerial structure to limit the length of the gap if the rail does break.

CWR is a standard now employed in the transit industry. Therefore, transit system designers must understand how it interacts with aerial structures as the temperature changes in order to provide a safe track and structure.

Expansion (sliding) rail joints are used in certain circumstances to reduce the interactive forces between the CWR and the structure. These include locations where special trackwork is installed on the aerial structure and where the aerial structure includes very long spans and or spans of extremely sharp curvature.

Rails can be attached to the structure in a variety of ways. The most common mechanism is the use of direct fixation fasteners with elastic spring clips. High-restraint rail clips have also been used in the vicinity of substructure units (piers and abutments) with fixed bearings, as well as adjacent to special trackwork. Also, zero-longitudinal-restraint fasteners can be installed to minimize the interaction forces between CWR and the aerial structure. A common configuration will employ high-restraint rail fastenings in a track zone centered above a “fixed-fixed” bearing location while low- or zero-restraint fastenings are used at intermediate zones straddling “expansion-expansion” bearings.

The decision concerning which type of deck construction to use with CWR has profound implications for construction cost. Based on the difference in cost of aerial structures with and without CWR, and the resultant thermal effects considered in the structural design, the most conservative design using CWR could increase structure costs by approximately 20%.[5] However, there are many variables to consider when choosing the type of deck to use on any particular transit structure.

7.5.3 Force Distribution between Rails and Superstructure

The thermal action in a direct fixation bridge exerts additional interactive axial forces and deformations on the rails and superstructure. Reaction loads are applied to the substructure

(piers and abutments) through the fixed bearings and by shear or friction through the expansion bearings. The aerial structure must also resist lateral components of the longitudinal loads on curved track. When the cumulative resistance of the fastening devices (rail clips) along a length of superstructure is overcome, the superstructure slides relative to the rail.

Since CWR is not able to expand or contract, temperature increases above the rail installation temperature cause compressive forces that could buckle the rail. Rail fasteners prevent buckling of the rail. Temperature decreases below the rail installation temperature cause tensile forces that increase the probability of a rail break (a "pull-apart"). A rail break not only results in a gap in the rail that could cause a derailment, it creates unbalanced forces and moments in the aerial structure. Rail breaks are discussed in further detail in Article 7.5.5.

Based on these thermal effects, there are three problems to address in the design of aerial structures with CWR:

- Controlling the stresses in the rail attributed to the different longitudinal motions of the rail and the superstructure because of temperature changes or other causes
- Controlling the rail break gap size and resulting loads into the superstructure
- Transferring the superstructure loads and moments into the substructure

A structural system is formed when CWR is installed on a direct fixation aerial structure. The major components of this system include the following:[6]

- Long, elastic CWR, with ends anchored in the track beyond the abutments
- Elastic rail fasteners that attach the rails directly to the superstructure
- The elastic superstructure
- Elastic bearings connecting the girders to the substructure
- The elastic substructure anchored to rigid foundations

There are a number of principal design factors that affect the magnitude of the interaction movement and forces between the rails and the structure, including the following:[10],[11]

- The composition of the girder material (steel or concrete), which will affect the expansion/contraction response to temperature changes
- The girder length and type (simple span or continuous), which will affect the magnitude of thermal movement that the rail fasteners must accommodate
- The girder's support pattern of fixed and expansion bearings from adjacent spans on the piers (refer to Article 7.5.3)
- The magnitude of the temperature change
- The rail fastener layout and longitudinal restraint characteristics, including these four concepts of fastener and restraint:
    - Frictional restraint developed in mechanical fasteners

- Elastic restraint developed in elastic fasteners
- Elastic restraint developed in elastic fasteners with controlled rail slip
- Elastic and slip fasteners installed in accordance with the expected relative movements between girder and rail: to control rail creep, install sufficient elastic fasteners near the fixed bearing; to provide full lateral restraint and minimal longitudinal restraint, install slip fasteners over the balance of the girder length

Depending on the method used to attach the rails to the structure, the structural engineer must design the structure for longitudinal restraint loads induced by the fasteners, horizontal forces due to a rail break, and radial forces caused by thermal changes in rails on curved alignments. Today's designer can use computer models to simulate the entire structure/trackwork system to account for variations in the stiffness of the substructure and the dissipation of rail/structure interaction forces due to the substructure's deflection (see Article 7.5.4).

The thermal force in the rail is calculated by the following equation:[4],[7],[8]

$$F_r = A_r E_r \alpha (T_i - T_o) \qquad \text{(Equation 1)}$$

where F_r = thermal rail force

A_r = cross-sectional area of the rail

E_r = modulus of elasticity of steel

α = coefficient of thermal expansion

T_i = final rail temperature

T_o = effective construction temperature of the rail

On horizontal curves, the axial forces in the rail and superstructure result in radial forces. These radial forces are transferred to the substructure by the bearings. The magnitude of the radial force is a function of rail temperature, rail size, curve radius, and longitudinal fastener restraint. Refer to **Figure 7.5.1** as well as other pertinent publications for the equation to calculate the radial rail/structure interaction force.

Various solutions have been implemented in an attempt to minimize the interaction forces caused by placing CWR on aerial structures, including use of the following:

- Ballasted track instead of direct fixation track (refer to Article 7.4.2)
- Zero-longitudinal-restraint fasteners (refer to Article 7.6)
- High-restraint fasteners near the structure's point(s) of fixity and low-restraint fasteners on the remainder of the structure
- A series of rail expansion joints and low-restraint fasteners to allow the rail to move independently of the structure; this requires highly restrained zones to transfer traction and braking forces to the structure

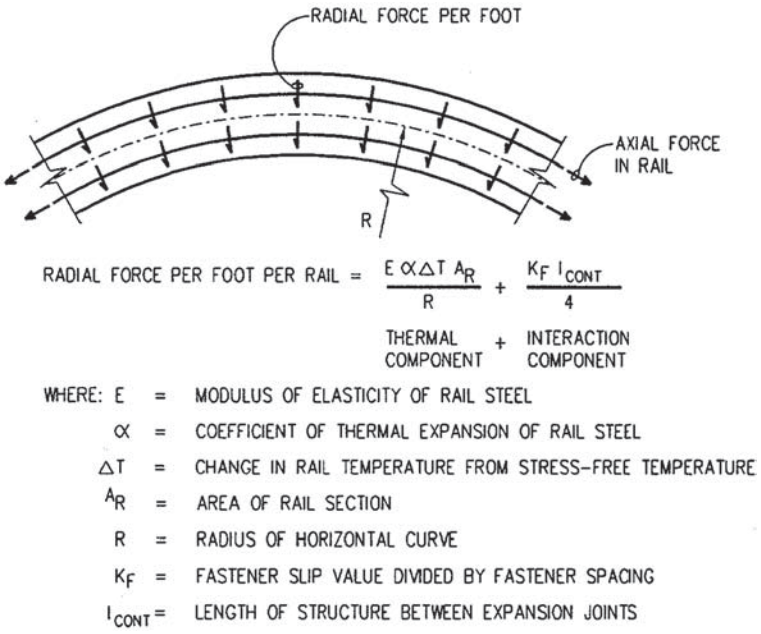


Figure 7.5.1 Radial rail/structure interaction forces[17]

7.5.4 Bearing Arrangement at the Piers

The magnitude of rail/structure interaction forces transferred to the substructure depends heavily on the bearing arrangement used. As shown in **Figure 7.5.2**, there are three commonly used bearing arrangements. Configuration A is a symmetrical bearing arrangement, with fixed bearings (or expansion bearings) from adjacent spans at the same pier. Configurations A and B are commonly used on modern transit systems that utilize CWR. Configuration C is a non-symmetrical bearing arrangement typically used on railroad and highway bridges.

As a guideline for rail transit systems with CWR, the symmetrical bearing arrangement is the most desirable. In this arrangement, the thermal interactive forces induced into the rail tend to cancel each other out at the piers. This is true as long as the adjacent spans are of similar length and geometry. On the contrary, if an expansion bearing at the end of one span is coupled with a fixed bearing at the end of the adjacent span on the shared pier (Configuration C), then the thermal interactive forces would have a cumulative effect.

Although the interactive forces at symmetrical bearing arrangements tend to cancel out before loading the piers, the structural engineer must still design the bearings and their anchor bolts to resist these forces.

7.5.5 Rail/Structure Interaction Analysis

Opinions differ within the transit design profession regarding the level of complexity required to design aerial structures subjected to thermal interaction forces from CWR. The interaction of the rails and supporting structure involves the control of rail creep, broken rail gaps, stresses induced

in the CWR, axial stresses induced in the guideway structure, and longitudinal and transverse forces developed in the supporting substructure.[8]

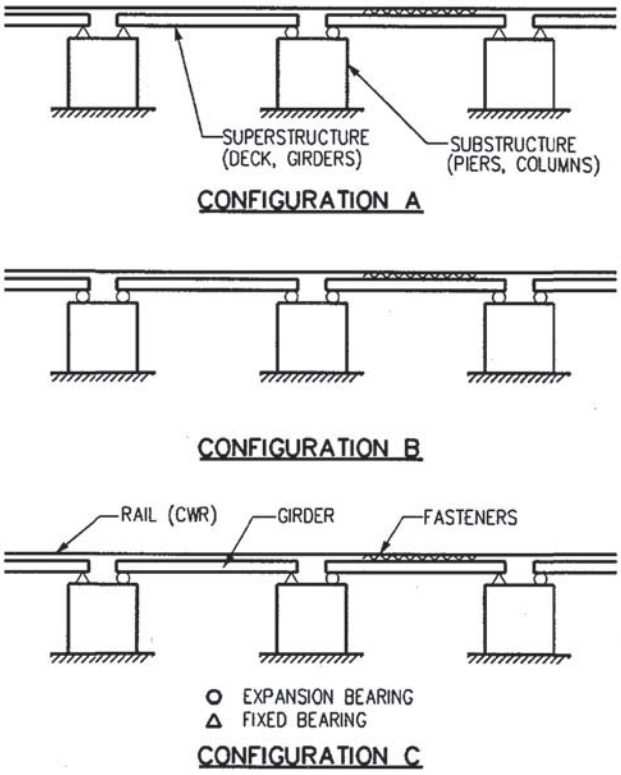


***Figure 7.5.2 Bearing configurations for elevated structure girders*[17]**

Some suggest that hand calculations are adequate and provide a good understanding of the important considerations of rail/structure interaction. Others have found that simpler analysis methods are unreliable in predicting stresses and structural behavior critical to significant CWR-related design elements.[5] Today's structural engineer has the advantage of being able to use computer software to more "exactly" analyze this complex interaction. The design elements that require investigation include the following:

- The control of stresses in rails attributed to thermally induced differential movements between the rail and supporting superstructure
- The control of the rail break gap size and the resulting loads transferred into the structure during low-temperature rail pull-apart failures
- The transfer of thermally induced loads from the superstructure through the bearings and into the substructure

The choice of the method used to analyze rail/structure interaction forces is clearly at the discretion of the experienced structural engineer. Depending on the length of the aerial structure

and other considerations, simple formulas may be used to determine the structural requirements. Alternately, complexities such as curved alignments, varying span lengths, and the type of structural elements may require that a rigorous three-dimensional structural analysis be performed. At times, the transit agency's design criteria will include the required analysis methodology.

Aerial structure alignments and geometrics are becoming more complex in order to fit into today's urban and suburban communities, and computer modeling and analysis techniques are becoming commonplace in the structure design environment. As a result, it is now typical for structural modeling to be part of the aerial structure design effort. Structural analysis models can vary widely depending on the structure's geometry and structural characteristics. In general, these analysis models consist of a network of beam-type elements and nodes that join or connect the different beam elements together. This skeleton or framework defines the critical structural elements and how they interact with each other and the track. The computer model allows the experienced structural engineer to mimic foundation conditions, model variations in structural stiffness, and apply forces to the structure in order to study the overall behavior and response of the structure. **Figures 7.5.3** and **7.5.4** depict the configuration of generic structural models used to analyze LRT aerial structures.

For additional information related to structural modeling techniques used for LRT aerial structures, see *Design Guideline for the Thermal Interactive Forces Between CWR and the New Jersey Transit LRT Aerial Structures, Hudson-Bergen Light Rail Transit System*. [35]

7.5.6 Rail Break/Rail Gap Occurrences

A rail break occurs when a thermally induced tensile force resulting from a significant decrease in temperature exceeds the ultimate tensile strength of the rail. The rail break is likely to occur at or near an expansion joint in the superstructure or at a poor quality weld, a rail flaw, or other weak spot in the rail.

The structure's expansion joint is a likely area where a rail break can occur because the girder's end rotations increase flexural stresses in the rail and the tensile stress already in the rail is likely to be at its maximum value at this location.[4],[7],[12]

A cold-weather broken rail on a rail transit bridge is an important consideration because of the potential to transfer a large eccentric force to the bridge and because a derailment might occur because of the resulting rail gap. For these reasons, aerial structure designers must consider the rail break condition. Limits on the size of the rail gap have to be established, usually based on the rail vehicle's wheel diameter. It is commonly assumed that only one rail of a single- or double-track alignment will break at any one time.

When the rail breaks, the direct fixation rail fasteners situated between the break and the thermal neutral point experience a sudden loading as the rail retreats from the point of the break. Then, the rail slips through the fasteners whose pads have deformed beyond their elastic limit, engaging enough fasteners to resist the remaining thermal force. Once a sufficient number of fasteners are engaged to balance the thermal force in the rail, the rail ceases to move.

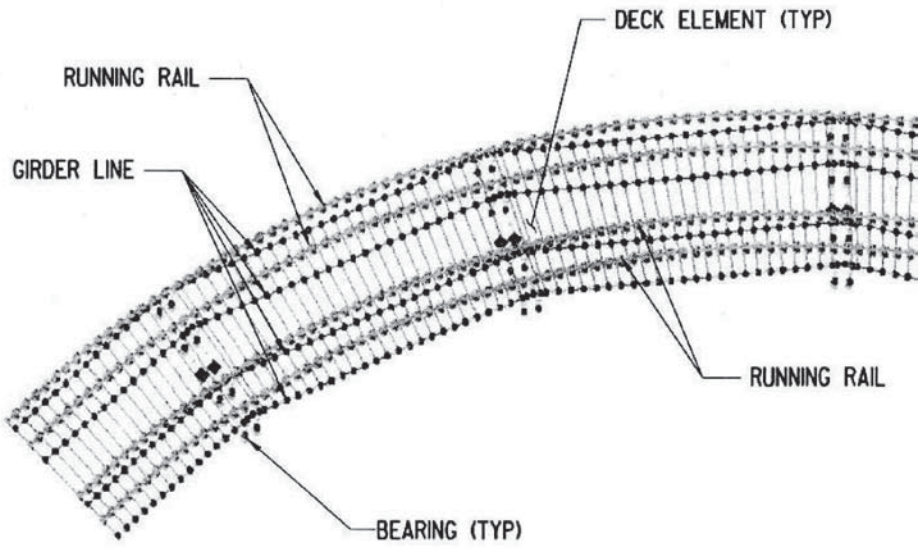


Figure 7.5.3 Typical structural analysis model

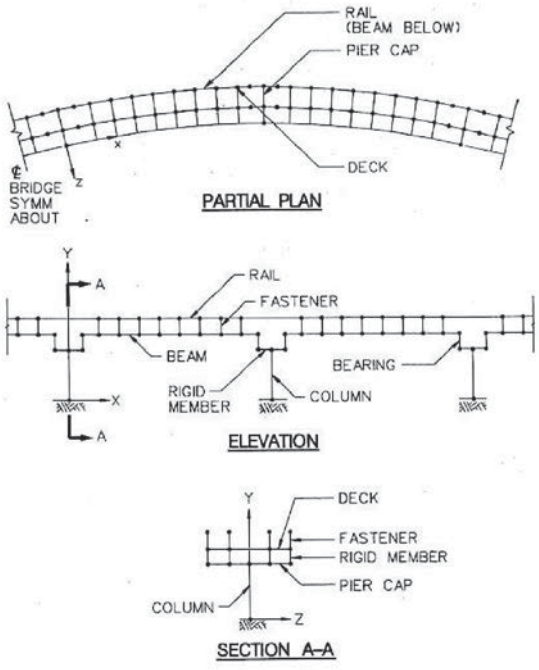


Figure 7.5.4 Typical structural model components

The unbalanced force from the broken rail is resisted by the other unbroken rail(s) and the aerial structure. The portion of the rail break force that is resisted by the unbroken rail(s) versus the aerial structure is significantly affected by the substructure's longitudinal stiffness (the force required to induce a unit deformation in a component), the bearing configuration, and the rail fastener's restraint characteristics.[5]

Refer to **Table 7.5.1** for a comparison of the rail gap size for different column stiffness and levels of fastener restraint. Note that progressively lower loads are transferred to the columns as column stiffness decreases. As a result, higher loads are transferred to the unbroken rails. This increases the thermally induced stress in this rail and raises the possibility of a second rail break. With higher restraint fasteners, more load is transferred to the unbroken rail and less to the column than is the case with medium-restraint fasteners.

Researchers have found that the superstructure's bearing arrangement, as discussed in Article 7.5.3, has little effect on rail gap size. Nonetheless, decreasing the fastener's longitudinal stiffness or slip force limit, or both, will result in an increased rail gap size.

The redistribution of the rail break force to the substructure causes a longitudinal deflection in the substructure. The resulting substructure deflection and the thermal slip of the broken rail combine to create the total gap in the broken rail.

Rail gap size is generally estimated using the following equation:[5]

$$G = 2\,(X_{C1} + X_{C2} - X_{C3}) \qquad \text{(Equation 2)}$$

where

G = rail gap, inches [cm]

$X_{C1} = P_{fns}/K_f$, the maximum longitudinal deflection of the non-slip fastener

$X_{C2} = \alpha\Delta T L_s$, the nominal rail contraction

$X_{C3} = (n_s P_{fs} + n_{ns} P_{fns})\, L_s/2A_r E_r$, the reduction in rail contraction caused by fastener constraint

and where the factors in the formulae above are as follows

α = coefficient of expansion, 6.5 x 10^{-6} in/in/°F [1.17x10^{-5} cm/cm/°C] for steel

ΔT = temperature change, °F [°C]

L_s = length of span (fixed to expansion point), inches [cm]

P_{fs} = minimum longitudinal restraint force in controlled-slip fastener, lb [kg]

P_{fns} = minimum longitudinal restraint force in non-slip fastener, lb [kg]

K_f = fastener longitudinal stiffness lb/in [kg/cm]

n_{ns} = number of non-slip fasteners in span

n_s = number of controlled-slip fasteners in span

A_r = cross-sectional area of rail (11.25 in^2 [72.58 cm^2] for 115 RE rail)

E_r = rail steel modulus of elasticity, 30 X 10^6 lb/in^2 [2.1 X 10^6 kg/cm^2]

Table 7.5.1 Effects of unbroken rail and column longitudinal stiffness on loads transferred to the substructure[5]

	Medium Restraint		High Restraint	
Column Stiffness (lb/in)	Load (lb)	Gap Size* (in)	Load (lb)	Gap Size* (in)
Rigid	131,000	0.67	134,000	0.79
500,000	50,600	0.89	35,800	0.89
100,000	17,700	1.17	11,600	0.96
40,000	9,300	1.27	5,800	1.15

* Assuming a symmetrical girder bearing configuration of E—F/F—E/E—F and a 60° F temperature drop.

A simplified form of Equation 2 has been used to estimate rail gap size, based on a length, L, on either side of the break over which full rail anchorage is provided, so that

$$G = (\alpha\Delta T)^2 \ A_r E_r / R_f \qquad \text{(Equation 3)}$$

where R_f is the longitudinal restraint per inch of rail in pounds per inch (kilograms per centimeter).

Equation 2 provides a reasonable estimate of rail gap size for medium- and high-restraint fasteners, but significantly underestimates the rail gap size for low-restraint fasteners. Low-restraint fasteners generally do not adequately control the size of the rail gap. Although Equation 3 provides relatively accurate estimates in many cases, it does not provide accurate estimates where high-restraint fasteners are used. Improved accuracy can be obtained with Equation 2 if the term X_{C2} is modified to use the estimated total number of fasteners over which the locked-in load is distributed. Therefore:

$$G = 2(X_{C1} + X_{C2} - X_{C3}) \qquad \text{(Equation 4)}$$

which appears identical to Equation 2 except that the X_{C2} factor is modified as follows:

X_{C2} = $0.5\ \alpha\Delta T\ n_x L_s$

n_x = $P_T/P_{fmax} = P_{fmax}K_r/2P_TK_f$

P_T = $\alpha\Delta T\ A_rE_r$, the thermal load, lb [kg]

P_{fmax} = $(n_{ns}P_{fns} + n_sP_{fs})/(n_{ns} + n_s)$, the average fastener restraint limit, lb [kg]

K_r = A_rE_r/L_f, the rail spring, lb/inch [kg/cm]

K_f = fastener longitudinal stiffness lb/inch [kg/cm]

L_f = fastener longitudinal spacing, inches [cm]

Equations 2 and 3 estimate rail gap size assuming linear load distributions. Typically, finite-element computer models show the fastener load distributions to be nonlinear. The computer modeling discussion provided in Article 7.5.4 applies to the rail break/gap analysis since the same structural model can be used to determine rail gap and loads resulting from the rail break condition. Refer to **Figure 7.5.5** for the rail gap sizes predicted using a finite-element model.

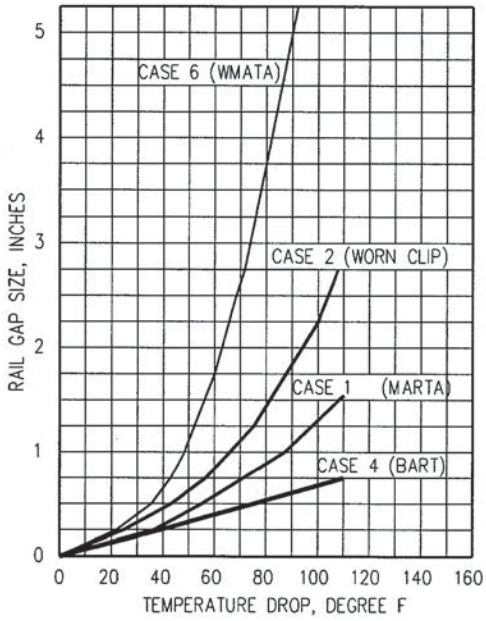


***Figure 7.5.5 Rail break gap size predicted by finite computer model*[5]**

Table 7.5.2 summarizes estimated rail gap size using different equations and software. Once the rail gap size has been estimated, the variables affecting the magnitude of the gap (such as rail fastener spacing and stiffness) should be adjusted to limit the size of the gap. This will minimize the chance of a rail vehicle derailment caused by a rail gap. The size of the rail gap is usually limited based on the diameter of the vehicle's wheel. Typically accepted rail gaps are in the range of 2 inches [50 millimeters] for a 16-inch [400-millimeter] diameter wheel.[4] Notably, wheels that small are seldom seen in rail transit vehicles, so there is a considerable factor of safety in limiting the rail gap to 2 inches.

It is interesting to note that efforts to control rail gap size offer opposing solutions. For safety reasons, the length of the rail gap should be minimized to reduce the possibility of a derailment. On the other hand, the forces and moments transferred to the structure due to a rail break should be minimized to achieve an economical structure. To resolve safety issues, fasteners with a relatively high longitudinal restraint should be used. To address the structural issues, fasteners with a relatively low longitudinal restraint should be used. The trackwork and structural engineers must coordinate the opposing design requirements to balance the needs for each structure.

Table 7.5.2 Comparison of rail break gap by different formulas[5]

Rail Break Gap Size Estimates (in)						
Case	Equation 2	Equation 3	Equation 4	TBTRACK[b] $\Delta T_g = 0$	TRKTHRM[b] $\Delta T_g = 0$	TRKTHRM[b] $\Delta T_g = 60$
1	0.69	0.62	0.83	0.55	0.74	0.67
2	0.72	1.23	1.35	0.85	1.29	1.31
3	0.89	1.23	1.55	0.97	1.38	1.47
4	0.51	0.15	0.99	0.40	0.50	0.79
5	0.57	0.62	0.68		0.66	0.63
6	0.85	2.29[a]	2.47	1.77	2.39	2.68
7	0.82	1.43[a]	1.62	—	1.54	1.61
8	1.21	0.68	1.44	—	1.20	1.14

Notes. ΔT_g = Temperature change in the girder; the girder bearing configuration = E-F/F-E/E-F; the length of the span = 80 ft; the length of the fastener = 30 in; and the temperature change in the rail = 60° F (temperature drop).

[a] Using average of $R_f = n_s P_{fs} + n_{ns} P_{fns})/(n_s + n_{ns})$ where n_s = the number of slip fasteners, and n_{ns} = the number of non-slip fasteners.

[b] TBTRACK and TRKTHRM are programs developed to calculate rail break gap size.

— Designates cases for which no data were published.

7.5.7 Terminating CWR on Aerial Structures

As much as possible, CWR should not be terminated on an aerial structure due to the large termination force transferred to the structure. Problems arise when specialwork must be located on an aerial structure due to the length of the structure, the needs of the transit operations, or other occurrences.

Unbalanced thermal forces exist in specialwork locations due to discontinuities in the rail. Standard turnout units, by design, transfer high forces through the units on an aerial structure, which causes misalignment and wear.[12] For these reasons, designers should avoid placing specialwork on aerial structures. When this cannot be avoided, there are ways to accommodate the specialwork without causing it to malfunction. To accommodate the large forces occurring at locations of specialwork, either rail anchorages or rail expansion joints can be used.

Rail anchorages (not to be confused with rail anchors—the snap-on device used to control rail movement in timber tie ballasted track) are large units that clamp onto the rail and anchor it to the plinth concrete below with no possibility of slippage. See Chapter 5, Figure 5.10.2, for a typical rail anchorage assembly. One of these units is placed in each rail on each approach to the special trackwork unit. Rail anchorages create a zero force condition through the specialwork, but pass the rail termination force to the underlying structure. However, the massiveness of the resulting substructure may be both aesthetically and economically undesirable.

Sliding rail expansion joints have been used both with and without the rail anchorages noted above. The following should be considered when using sliding rail expansion joints:

- The construction length of the sliding rail joints
- The length of structure required to accommodate the specialwork and sliding rail joint
- The design, location, and installation details of the rail anchors

As an alternative to rail anchors, some transit systems have used a tie bar device to accommodate specialwork on their aerial structures. Tie bars carry the CWR stresses around the special trackwork unit at deck elevation rather than transferring those loads down to the substructure. See **Figure 7.5.6** for a picture of a tie bar installation at an aerial structure crossover.

With a tie bar system, the CWR is interrupted at the crossover and the rail ends are attached as rigidly as possible to special "AXO" girders adjacent to the outer ends of the specialwork. The AXO girders are similar to standard girders except for the addition of an embedded steel plate to which the tie bar is attached by welding. The tie bar, a structural steel member with a cross section equal to two rails, is located on the centerline of each track and is welded to the embedded plates on the centerline of the two AXO girders. The tie bar rests on Teflon bearing pads placed directly on the concrete deck for the length of the crossover.

When the temperature changes, the thermal force built up at the end of the CWR is transferred to an AXO girder through a group of rail fasteners equally spaced along the girder. An equal and opposite thermal force is developed in the tie bar and transferred to the AXO girder through a welded connection. Therefore, the net longitudinal thermal force is directed through the tie bar instead of the through the piers or the specialwork, where the trackwork could be damaged.

One problem with all of the methods discussed above is making certain that the rails remain electrically isolated from both the superstructure and each other. If electrical isolation is compromised, signal system failures and stray currents could result. However, the mechanical/structural properties of the dielectric materials used to isolate the rails may not accommodate the stresses associated with the absolute termination of the CWR. Avoidance of special trackwork on aerial structures is quite likely the only way to fully mitigate this concern.

7.6 DIRECT FIXATION FASTENERS

Since the majority of transit properties now use CWR with direct fixation deck construction, the aerial structure designer should understand the types of rail fasteners presently available. Direct fixation rail fasteners secure the CWR to the deck of the aerial structure. The bottom portion of the fastener is bolted to the deck (either directly or through plinths), and the top portion is bolted or clipped to the bottom flange of the rail.

Low-restraint, moderate-restraint, and high-restraint fastener rail clips are available. In addition, some transit properties have utilized zero-longitudinal-restraint (ZLR) fasteners in certain circumstances. Although ZLR fasteners allow the superstructure to move longitudinally without generating thermal interaction forces, the rail gap size at a rail break has to be carefully considered when they are used.

With a conventional direct fixation fastener, the elastomer provides isolation of the high wheel/rail impact forces from the deck, electrical isolation, vertical elasticity to dampen noise and vibration, longitudinal elasticity to accommodate rail/structure interaction movements, and distribution of the wheel loads longitudinally along the rail. The fastener also provides full restraint in the lateral direction, maintains the desired rail tolerances, and prevents rail buckling under high temperature.

The level of longitudinal restraint chosen for the fastener is a compromise between the restraint required to limit the rail gap size and the desire to minimize rail/structure interaction forces.[6],[8]

(Photos courtesy of Bay Area Rapid Transit)[6]

Figure 7.5.6 Tie bar on aerial crossover

The following are typical ranges of direct fixation fastener properties:

Vertical fastener stiffness:
75,000 to 150,000 lb/in (13,300 to 26,600 N/mm)

Lateral fastener stiffness:
22,000 to 64,000 lb/in (3,900 to 11,400 N/mm)

Longitudinal fastener stiffness
3,400 to 18,000 lb/in (600 to 3,200 N/mm)

Longitudinal restraint
2,000 to 3,500 lb (9,000 to 15,750 N)

Direct fixation fasteners are commonly spaced at 30 inches [762 millimeters] on center. Other spacings may be required if determined by analysis of rail bending stresses, interaction forces of the rail and rail fasteners, and the rail gap size at a rail break location. Trackwork and structural engineers need to carefully coordinate fastener spacing on sharply skewed bridges to ensure that the fasteners are adequately supported on each side of the joints in the deck.

Another key consideration to be coordinated between the trackwork engineer and the structural engineer is structure vibration. The structural engineer typically determines the unloaded (i.e., without live load) natural frequency of the aerial structure. Then the spacing and characteristics of the rail fasteners can be chosen in an effort to control resonance. It is common for transit agencies to specify structure vibration limitations in the aerial structure design criteria. These limitations are established in an effort to minimize the potential dynamic interaction between the structure and the LRVs. Further discussion of structure vibration and the role of direct fixation rail fasteners in mitigating vibration can be found in Chapter 5 and Chapter 9 of this Handbook.

7.7 TRACK SUPPORT SLABS-ON-GRADE

It is common to have slabs-on-grade along many portions of a LRT project corridor. Typically this slab is also functioning as concrete pavement for use by rubber-tired vehicles operating in the same lanes as the LRV, but it can also include slab track in exclusive or semi-exclusive guideways such as direct fixation track and some forms of grass track. The design, analysis, and detailing of slabs-on-grade are important to the durability and performance of the slab. An investigation of the approaches used to design and detail slabs-on-grade used by various transit agencies shows how methodologies and detailing practices vary considerably.

The basic analysis of the slab-on-grade should typically involve performing a classical beam-on-elastic foundation analysis. This analysis will consider the effects of subsurface conditions, geotechnical design parameters (subgrade modulus), and the anticipated LRV loads. For instance, the stiffness or “softness” of the subgrade will in most cases dictate the relationship between the slab thickness required and the amount of flexural reinforcement needed. If a very stiff subgrade is present, minimal bending would be expected to occur in the slab, thus resulting in minimal reinforcement. Conversely, if loose and/or compressible soil exists, the tendency for the slab to differentially deflect and flex will be high, thus warranting a thicker and/or more heavily reinforced slab.

The presence of steel reinforcing in slabs-on-grade used to support LRV traffic raises concerns related to stray current and its mitigation. Questions arise related to whether the slab reinforcing attracts or deters stray current. Additionally, consideration needs to be given as to whether the reinforcing in these slabs should be epoxy coated or uncoated and electrically bonded or not. These issues need to be coordinated during the design process in order to provide durable, long-lasting slabs. Chapter 8 provides additional information related to corrosion control and stray current mitigation.

Four different configurations of slabs-on-grade are common to LRT projects. These include the following:

- Single-pour embedded slabs, in which the rails are embedded in the slab.
- Two-pour embedded slabs, in which the rails sit on top of the first pour and, after being checked for proper alignment, are embedded with a second pour. A horizontal construction joint is located at the base-of-rail elevation.
- Slabs supporting an open, direct fixation trackform.
- Slabs supporting a grass track detail.

In general, the slab-on-grade designer will need to consider the following items during the design phase, regardless of which type of slab is used:

- The structural engineer may be the lead designer of the slab-on-grade, but this engineer will seek input from the trackwork engineer, geotechnical engineer, roadway/pavement engineer, and corrosion engineer in order to confirm an efficient design for the slab-on-grade.
- The trackwork engineer will be consulted to confirm the details of the rail troughs or rail embedments to be formed in the slab-on-grade. The details of the waterproofing system, stray current isolation system and rail fastener and anchorage systems are critical in the design of the slab-on-grade.
- Subsurface data, including the engineering properties of the subgrade material, are required for the structural analysis to determine the thickness and reinforcing requirements for the slab. The subgrade modulus of the soil beneath the slab is critical to the elastic foundation analysis that is typically performed. Some transit properties include minimal, or even no reinforcing steel in the support slabs while other systems, projects, or specific locations on a project require that a slab be heavily reinforced. The stiffness and uniformity of the subgrade materials are key items in determining the thickness of the support slab and whether or not it needs to be reinforced.
- The presence of utilities beneath the slab-on-grade needs to be confirmed at the early stages of design. A decision needs to be made regarding the methods by which the utilities will be serviced, maintained, and replaced if necessary as they pass below the track slab. On some projects, this issue has led to a decision that the track slab needed to be able to span across an open utility trench without disrupting LRT revenue service, resulting in an extremely heavily reinforced slab. On one such project, a utility repair was undertaken several years later; however, it is questionable whether the utility contractor's task was actually made easier by the bridging slab. More conventional methods, such as pipe jacking or directional boring, which do not require a bridging slab, may have been just as efficient and could have saved the rail project a substantial percentage of the original embedded track construction

cost. At least two legacy streetcar systems use no reinforcing steel at all in their embedded tracks. A major water main break directly beneath one such track was repaired without damage to the track as the track itself was sufficiently stiff to bridge over the excavation. Streetcar service was interrupted for a short time while the repair was in process, and shuttle buses were substituted over the affected portion of the route.

- The roadway/pavement engineer will review the slab-on-grade design from the perspective of supporting the loads from rubber-tired vehicles. An analysis with the wheel loads placed along the edges of the slab-on-grade may be critical to confirming the adequacy of the slab design in those areas. Obviously, the LRVs can only travel along the rails embedded in the slab, but the rubber-tired vehicles can travel in any direction.
- Based on the details of the stray current isolation system surrounding the rail in the trough, the structural engineer will consult with the corrosion engineer to determine the need for either epoxy-coated rebar or electrically bonded rebar in the slab. In addition, the corrosion engineer will evaluate the depth of the slab and its proximity to underground utilities to assist the structural engineer in determining a cost-effective slab design.

The trackwork engineer and the structural engineer must coordinate on which slab type is best suited for a particular application on a site-specific basis.

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Chapter 8—Corrosion Control

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CHAPTER 8—CORROSION CONTROL

8.1 INTRODUCTION

Electrified rail transit systems, both light and heavy rail, typically utilize the track system as the negative side of the electrical circuit in the system's traction power network. In light rail transit systems, the positive side, which carries direct current (DC) electrical current from the substation to the transit vehicle, is typically an overhead contact wire system or catenary. Because perfect electrical insulators do not exist, electrical currents will leak out of this circuit and escape into the soil to find the path of least resistance back to the substation. The amount of such stray currents will be inversely proportional to the efficacy of the electrical insulation provided and directly related to the conductivity of the soil and any alternative current paths back to the substation such as pipes, cables, reinforcing steel, etc. Where this current leaves a conductor to jump to another, corrosion occurs by electrolysis.

In addition to stray current that originates at an electrified rail transit line, other sources of stray current can be found in virtually any metropolitan area. It is important to identify these so that the transit line isn't blamed for all corrosion. For the same reason, regular monitoring of the structures for stray current and corrective maintenance of any leaks are both extremely important.

While corrosion can occur for many reasons, stray current is responsible for a large proportion of the corrosion damage that occurs on electrified rail transit systems. Controlling that damage requires controlling the stray current at its source—the track structure. The track engineer, working together with other disciplines, obviously plays a key role in that mission.

8.1.1 A Primer for LRT Stray Current

In 1967, Mars G. Fontana, a professor at The Ohio State University, published *Corrosion Engineering*, one of the seminal textbooks on corrosion. In it, he stated:

> *The term stray current refers to extraneous direct currents in the earth. If a metallic object is placed in a strong current field, a potential difference develops across it and accelerated corrosion occurs at points where current leaves the object and enters the soil. Stray current problems were quite common in previous years due to current leakage from trolley tracks. Pipelines and tanks under tracks were rapidly corroded. However, since this type of transportation is now obsolete, stray currents from this source are no longer a problem.* [1]

That text was representative of its times since, by the mid-1960s, streetcars had been eliminated from all but a handful of North American cities. However, since then, "trolley tracks" have evolved into light rail transit (LRT) lines, reintroducing the possibility of stray currents from LRT operations and the attendant corrosion problems.

Typically, unless a fault has occurred in an insulator, stray currents from the positive side of the light rail transit traction power circuit (e.g., the catenary) are minuscule. Stray currents from the track, on the other hand, are common and can get quite large due in no small part to the proximity of the track to the ground. Once in the soil, stray currents will follow any available conductor to

get back to the traction power substation. These paths can include the soil itself; buried utility pipelines and cables; and other metallic structures, such as bridges, along the way. If an alternative path offers less electrical resistance than another route, then the better conductor will carry proportionally more of the stray current. In extreme examples, particularly when the electrical continuity of the track structure is poor, more electricity will return as stray current than through the running rails. Some older elevated railway systems were actually designed for this occurrence.

The problem with stray currents evolves from the fact that whenever electric current leaves a metallic conductor (i.e., a water pipe) and returns to the soil (perhaps because it is attracted to a nearby gas line), it causes corrosion on the surface of the conductor it is leaving. This is the same phenomenon that occurs when a metallic object is electroplated, such as when construction materials are zinc plated. In the case of transit system stray currents, the typical current path can involve several different conductors as the electricity wends its way back to the substation; therefore, corrosion can occur at multiple locations. This can create conditions that range from leaking water lines to gas line explosions. The rail itself will also corrode wherever the current jumps from it to reach the first alternative conductor. Structures along the transit line, particularly steel bridges and embedded reinforcing steel, are also at risk. Hence, multiple parties have an interest in controlling or eliminating the leakage of stray currents and minimizing the damage they inflict.

Stray currents are common on a light rail transit system because its track structures are typically close to the ground. Grade crossings, embedded track, and fouled or muddy ballast are common locations for propagation of stray currents. Because of the maze of underground utility lines typically found in urban and suburban areas where light rail transit systems are built, abundant alternative electrical paths exist. Predicting the likely path of potential stray currents and defining methods to protect against them can be extremely complex. Because of this complexity, it is essential that the advice of a certified corrosion control specialist with stray current experience be sought from the beginning of design.

One case study was conducted along a segment of an operating DC streetcar line where significant corrosion of a parallel water main was being experienced. The study evaluated the statistical relationship between rail volts and water main volts. The methodology involved the use of digital recording instruments and spreadsheet software to perform the required data acquisition and analysis. Under normal operating conditions, a correlation coefficient of 94% was calculated. A control measurement was obtained with the trolley line out of service that resulted in a correlation coefficient of only 3%, indicating that the operation of the railway had a significant impact on the corrosion of the water main. The old bolted joint rails, which were embedded in the street and over the centerline of the water main, were then replaced with continuously welded rail that was insulated with a rail boot. Post construction measurements showed that the correlation coefficient under normal operations had reduced approximately 10-fold—from 94% to 9%.

Some of the principal measures that can be taken to minimize traction current leakage include the following:

- If jointed track is used, install electrical bonding across the joints. One of the many advantages of continuous welded rail (CWR) is that it offers a superior traction power return.

- Insulate rails from their fastenings and encase rails in embedded track in an insulating material.
- The steel reinforcement in the underlying concrete slab can be continuously welded / cross bonded. This will eliminate the electrical potentials that can develop between individual reinforcing steel bars and lead to corrosion.
- In ballasted track areas, the ballast should be clean, non-conductive, well-drained, and not in contact with the rail. (This condition is often difficult to achieve and maintain in practice, particularly in the vicinity of highway grade crossings.)
- Conduct corrosion surveys on susceptible metal structures both before commencing design of the system and again prior to the commencement of revenue service train operations. These surveys provide baseline information on any stray currents that pre-exist in the light rail system and benchmark the efficacy of the measures taken during design and construction. Subsequently, regularly scheduled monitoring for stray currents as part of a preventative maintenance program is also very important so that leakage problems can be identified and corrected before significant damage occurs.
- Provide auxiliary conductors to improve and supplement the electrical capacity of the rail return system. This can be accomplished by cross bonding connections between all rails of the track(s) and/or by adding supplemental negative return cables.
- If the bonded reinforcing steel mentioned above is bonded back to the negative side of the traction power substations, it can collect stray currents that get past the primary insulating system. (This is an extreme measure that should not be undertaken without extensive investigation of the alternatives.)

Existing pipes and cables in the vicinity of the tracks must be investigated and protective action taken as necessary to protect them from stray current corrosion.

Whether the light rail operator or the local utility takes responsibility, it is imperative that strategic action is undertaken to mitigate the effects of stray current corrosion in the design phase and during construction. This will avoid corrosion from becoming a costly and dangerous maintenance issue later.

8.1.2 The Interdisciplinary Corrosion Control Team

An interdisciplinary approach is necessary to determine the required design for stray current mitigation. Recommendations are developed in coordination with all other rail system disciplines including track, traction power, signals, and communications systems, as well as facility infrastructure and systems such as civil, structural, electrical, mechanical, and plumbing. Corrosion control measures are needed for all those project elements, even though, at first glance, they seem entirely unrelated to traction power and stray traction power current. A few examples of the stray current problems that can arise due to the actions of other disciplines include the following:

- Signals—“sneak paths” for stray current can occur on circuitry provided for broken rail detection.

- Track—poor drainage and the resultant fouling of ballast can bypass insulation measures and result in localized corrosion of rails and rail fastenings.
- Communications—stray currents can sometimes occur over cable shields.

8.2 TRANSIT STRAY CURRENT

8.2.1 Stray Current Circuitry

Traction power is normally supplied to light rail vehicles (LRVs) by a positive overhead contact wire system. The direct current is picked up by a vehicle pantograph to power the motor and then returns to the substation via the running rails, which become the negative part of the circuit. Unfortunately, a portion of the current strays from the running rails and flows onto parallel metallic structures such as reinforcing steel; utility pipes and cables; and other structures such as pilings, ground grids, and foundation reinforcing bars.

8.2.2 Stray Current Effects

Corrosion of metallic structures is an electrochemical process that usually involves small amounts of direct current. It is an "electro" process because of the flow of electrical current. It is a "chemical" process because of the chemical reaction that occurs on the surface and corrodes the metal. Faraday's Law relates the amount of metal lost in an electrolytic process to the electric current generated from that metal. For this reason, a unidirectional DC current along a buried utility line, such as a water line, will drive metal from the pipe when subjected to stray currents over a period of time. One ampere of direct current flowing for 1 year will corrode 20 pounds [9 kg] of iron, 46 pounds [21 kg] of copper, or 74 pounds [36 kg] of lead. Natural galvanic corrosion involves only milliamperes of current, so many buried structures can last several years or decades without structural distress or failure.

Unlike the very small currents associated with galvanic corrosion, stray current corrosion from a transit system can involve the continuous flow of several hundred amperes. The same physical laws apply for corrosion of the metal, electron flow, chemical reactions, etc., but metal loss is much faster because of the larger amounts of current involved. For example, with 200 amperes of current discharging from an underground steel structure, 2 tons [1.8 tonnes] of metal will be corroded in 1 year:

20 pounds per ampere per year x 200 amperes = 4,000 pounds of steel corroded

[9 kg per ampere per year x 200 amperes = 1,800 kg of steel corroded]

If this current flow is concentrated at one location, structural failure can occur in a very short time period. Even if the leakage is spread out over a length of track, stray current from a light rail system will still aggressively corrode transit rails, rebar, and steel structural members and all adjacent underground metallic structures. While various methods are available for mitigating this issue, the most effective method is to control the current at its source, i.e., the surface of the rails.

Faraday's law cannot be applied to AC stray currents since they are not unidirectional and alternate between positive and negative polarities.

8.2.3 Historical Background

The phenomenon of stray currents from electrified street railways was observed almost immediately when the first electric trolley lines were constructed in the 1880s. The importance of maintaining good electrical continuity of the rails was quickly recognized, and many trolley systems welded rail joints 60 years before the process was widely accepted on “steam” railroads. Where rails could not be welded, they were electrically bonded to each other with copper cables. These measures reduce stray currents, but cannot eliminate them. No matter how good a conductor the track system is, some portion of the traction power current will always seek an alternative path back to the substation.

Utility companies fought this problem, both in the courts and in the field. Once the legal issues were resolved, the most effective means of minimizing stray current damage that was available in the late19th century was to make the buried utility network as electrically continuous as possible. Copper bonds were placed around joints in buried pipes and crossing utility lines were electrically bonded to each other. Finally, the entire utility network was directly connected to the negative bus of the traction power substation by “drain cables” so that any stray currents could return without causing significant corrosion along the way. All big city utility companies, plus the local streetcar company, participated in a “corrosion control committee” with the objective of ensuring that all new facilities were properly integrated into the system, thereby preserving the delicate balance of the network. (In many cities, a single holding company might own most of the utility companies and the trolley company as well; thus, such committees were not necessarily combative congregations.) Such methods were generally effective; however, a side effect of the improved underground electrical continuity was that the utility grid typically became better bonded than the track structure. As such, a significant portion of the traction power current would perversely elect to stray from the rails and use the buried utilities to get back to the substation. Where such currents left the track to seek the alternative paths, significant corrosion would occur at the base of the rails. It was not uncommon that track maintenance work would reveal that the base of rail had corroded away completely where it crossed above a particularly attractive utility line.

When trolley systems were abandoned in most cities, the corrosion committees were disbanded and the utility companies became less zealous about bonding their networks. In many cases, the introduction of non-metallic piping created significant electrical discontinuities in utility systems. Such gaps were of no consequence in a city without a local originator of significant levels of stray currents and their associated corrosion issues. With no trolley network in the neighborhood, incidental corrosion potential could typically be neutralized using sacrificial anodes. However, if a light rail system is introduced or reintroduced into such a city, sacrificial anodes are insufficient as they could corrode away in an extremely short time. The result can be corrosion problems not unlike those that occurred 100 years ago, with stray currents leaping off metal pipes when they reach an electrical dead-end at a non-metallic conduit or cross another potentially attractive conductor.

Reverting to the continuous utility bonding and drain cable methods of the past is typically neither a practical nor completely effective methodology of achieving stray current control. Because of the widespread use of non-metallic buried pipe and the subsequent high expense of recreating an

electrically continuous path through the utilities, it is typically much cheaper—and arguably easier—to attempt to effectively insulate the track structure from the ground so that stray currents are minimized from the beginning. Such insulation, coupled with other protective measures, including very selective bonding of utilities and drain cabling, is the foundation of stray current corrosion control measures on modern light rail transit systems. This controlled approach also protects rails and other transit structures that would be subjected to these stray currents.

8.2.4 Design Protection Components

8.2.4.1 Traction Power

Since the 1960s, increased efforts to reduce stray currents have been made through modifications to traction power substations. Typical modern substations are either ungrounded, "floating" above ground potential, or are grounded only through diodes that prevent stray currents from passing from the ground to the negative bus. This frequently reduces stray currents from hundreds of amperes to near zero. Completely ungrounded systems exhibit the greatest improvement in stray current control. Nevertheless, stray currents are still possible in an ungrounded system as it is entirely possible for current to leak out of the track at one spot, travel along alternative paths in the ground, and then return to the track at another location. Since the track itself must eventually be directly connected to the negative substation bus, stray currents can still occur and cause damage en route back to the traction power substation (TPSS).

8.2.4.2 Track and Structure Bonding

Achieving electrical continuity of the track structure is of paramount importance in keeping negative return current in the rails. The use of continuously welded rail, together with the installation of bonding cables around unavoidable bolted joints, provides most rail transit systems with an excellent current path through the rails. Stray current corrosion of transit structures can typically be controlled through electrical bonding. Since the 1960s, it has been common practice to also bond reinforcing steel in concrete structures so as to provide a continuous electrical path. The bonding is typically concentrated in reinforcing bars in the lowest portions of the structure and those surfaces in contact with the earth such as retaining walls.

Bonded reinforcing steel networks can provide a shielding effect for outside utility structures. Many light rail systems have been built with heavily reinforced slabs beneath the track to provide both structural support and a barrier against migration of stray currents into the ground.

8.2.4.3 Drain Cables

Stray current drain cables are common on legacy rail transit systems that predate the development of effective methods of electrically isolating the tracks. Drain cables are sometimes provided for future use on modern light rail systems, but are not necessarily connected to the utility system. Utility companies monitor their pipelines for any stray currents and, if problems are detected, the companies have the option of connecting to the drain cable as a last resort. Coupled with other protective systems, such cabling provides a secondary approach to corrosion protection in the event that the primary measures are ineffective at locations where excessive leakage from the rails occurs. However, since drain cables effectively decrease the resistance along a negative return path through the utility grid, their indiscriminate use can actually increase the amount of traction power current that leaves the rails. Therefore, drain cables should only be

used as a last resort and only after extensive investigation of the problems and alternative solutions.

Even if utilities are not connected to them, drainage cables are often used to provide an electrically continuous path between metallic components of the transit infrastructure (such as reinforcing steel in bridges and tunnels) and the negative side of the traction power substation. These drain cables protect those items from stray current corrosion. As a secondary benefit, the drain cable can prevent build-up of an electrical charge on the protected item, which could otherwise be hazardous to employee safety.

8.2.4.4 Trackwork

Ultimately, electrical insulation of the track structure offers the first line of defense against stray currents. Keeping the rails clean and dry is important, as are good insulators between the rail and the ties. Good storm water drainage is also critical since slowly moving runoff can deposit conductive sediments that can bypass insulators. Rails laid in street pavements need to have insulating coatings to maintain electrical isolation. Since track design is the focus of this Handbook, track insulation will be discussed in detail in Article 8.4. It must be emphasized, however, that providing track insulation alone is not a panacea, particularly if the track insulation systems are not regularly maintained and cleaned. If track insulation systems are compromised, for example by fouled ballast or dirty insulators, stray current leakage is inevitable. Thus, the required level of maintenance should be considered during design.

8.2.4.5 AC versus DC Considerations

It is important to note the distinction between the effects of stray currents from alternating current (AC) versus direct current (DC) sources. Stray currents that are generated by the DC electric traction system may cause severe corrosion to metallic components of the infrastructure. It is for that reason that the return components of DC electric traction systems are properly isolated from the ground. Alternating currents, on the other hand, cause cyclic anodic and cathodic conditions to occur at the same point as the current alternates from positive polarity (which is characterized by the discharge of current via the loss of metal ions into the surrounding media and which constitutes an anodic site) to negative polarity (which is characterized by the pickup of current via the plating of metal ions from the soil and which constitutes a cathodic site). It can be stated that the metal is corroded on the anodic or positive half-cycle, when current is discharged as metal ions, and protected on the negative or cathodic half-cycle, when current is received as metal ions are plated back onto the metal substrate.

It should be noted that the cathodic or negative half-cycle provides protection of the metal against corrosion since current is flowing into the metal rather than discharging from the metal as metal ions. This principle is referred to as cathodic protection and is a major component of corrosion control for metallic structures. While corrosion caused by AC current can occur due to the alternating positive and negative half-cycles, it is normally not significant in comparison with corrosion caused by DC current. It is therefore necessary for an LRT design team to know whether the measured values of stray currents detected during a baseline survey are in fact corrosive DC stray currents or AC ground currents from electric utility operations. This can have a significant impact on the prescribed mitigation measures and hence the project cost.

The identification of AC versus DC components of stray current should be considered in the determination of stray current mitigation requirements. It is not unusual for both DC traction power return currents and 60-Hz AC electric utility ground currents to be present in a given measurement of stray current activity. Sampling rates that are adequate to identify the various sources may be applied to identify the fundamental and harmonic frequencies of the AC components although the DC component, which is represented by the average value of the waveform, is of primary concern for purposes of stray current mitigation. Measurement techniques that do not permit the determination of the relative contributions of AC versus DC components can result in an incorrect quantification of DC stray current levels, falsely attributing a corrosive condition to AC stray current. This, in turn, could result in unwarranted recommendations for stray current mitigation and unnecessary cost to the project. It is, therefore, necessary that the LRT designer understand the relative contributions of AC versus DC components in order to specify corrosion control measures that are commensurate with the level of exposure.

8.2.4.6 Effect of Grounding and Bonding on Corrosion Control

The grounding and bonding of the infrastructure is intended to minimize touch and step potentials by providing a low resistance for current flow under both normal and fault conditions and hence reduce the voltages to which personnel can be exposed to within tolerable limits. The unintentional flow of electric traction power return currents over non-current-carrying metallic components of the infrastructure is in conflict with the intent of the grounding and bonding requirements of publication *NFPA 70: National Electrical Code* (NEC). The NEC is published by the National Fire Protection Association (NFPA). Article 250.6(A) of the NEC states:

> *The grounding of … normally non-current-carrying metal parts of equipment shall be installed and arranged in a manner that will prevent objectionable current.*[6]

The word “normally” in the quote above was added in the 2008 edition to acknowledge that fact that non-current-carrying metallic components that do not carry current under normal conditions may carry current under fault conditions until the fault is cleared. It should be noted that railroad systems are not within the scope of the NEC, as indicated in the following excerpt from Article 90.2(B) *Not Covered(3)*:

> *Installations [on] railways for generation, transformation, transmission, or distribution of power used exclusively for the operation of rolling stock or installations used exclusively for signaling and communications purposes.*[6]

Railroad traction power, signals, and communications systems are therefore not within the scope of the NEC. However, railroads typically invoke the NEC for various aspects of railroad installations and electrical installations at passenger stations, operations and maintenance facilities, control centers, and other facilities that represent places of assembly and occupancy classes where the safety of employees and the public is an issue. Since those facilities are normally grounded and bonded in accordance with NEC requirements, the flow of objectionable currents from the electric traction return system may occur and must be considered in the grounding and bonding plan.

It should be noted that the NEC is purely advisory from the perspective of the NFPA but may be invoked by an "Authority Having Jurisdiction" (AHJ), such as a local building code enforcement officer, as a legally required standard.

The provisions for safeguarding of persons from hazards arising from the installation, operation, or maintenance of conductors and equipment in electric supply stations and overhead and underground electric supply and communications lines are defined in publication IEEE C2—National Electrical Safety Code (NESC), which is published by the Institute of Electrical and Electronic Engineers, Inc. (IEEE).[7] Grounding rules make up a significant portion of that standard. In 1970, the section of the NESC entitled "Rules for the Installation and Maintenance of Electric Utilization Equipment" was deleted since those rules are now largely covered by the National Electrical Code. The following points are noted:

- The NESC covers utility facilities and functions up to the service point.
- The NEC covers utilization wiring requirements beyond the service point.

The NESC specifically includes railways, except for rolling stock, in its definition of utilities. The grounding requirements of the NESC are given in Section 9, "Grounding Methods for Electric Supply and Communications Facilities." However, the scope of the grounding requirements specifically excludes both the grounded return of electric railway traction power currents and lightning protection wires that are normally independent of supply or communications wires or equipment. The grounding of static wires that serve the dual role of lightning protection for the OCS contact wire assembly and electric traction return are therefore not covered by the NESC.

The NEC and NESC are voluntary standards but have been adopted in whole or in part by various states and local jurisdictional authorities. The determination regarding the legal status of the NEC and NESC in any particular state or locality is made by the AHJ. The LRT design team needs to identify the AHJ for their project and evaluate the impact of federal, state, and local laws.

8.3 TRACK ALIGNMENT AND TPSS LOCATION FACTORS

LRT routes that are circuitous instead of generally straight can induce stray currents as well. This is because the current may leave the track at one bend in the route, cut across several city blocks using the utility grid, and then jump back onto the rails at another point. LRT route decisions are typically made during early planning and environmental studies, and there often will be no corrosion or traction power engineers on the project staff at that time. The track alignment engineer may be one of the few engineering professionals involved during planning studies. By notifying the project planners that certain routings might have a greater or lesser propensity to instigate stray current problems, the track alignment engineer can possibly save the project much grief and cost at a later date.

Traction power engineers also play a key role in minimizing the propensity of return current to stray. Possibly the most important of these is the location and frequency of the traction power substations (TPSSs). It is highly desirable that each TPSS be located immediately adjacent to the route and generally no more than a mile apart along the route. TPSSs that are located at a

distance from the route of even a few city blocks can increase the propensity for stray current to take a "shortcut" back to the substation. This is one reason that TPSSs are preferably floating above ground potential—thereby forcing any stray current to return to the rails, as they will be the only route back to the substation.

8.4 TRACKWORK DESIGN

LRT systems utilize direct current electrical power to propel the vehicles. This current arrives at the vehicle via the OCS and is normally returned from the LRV to the substations through the rails. While many modern light rail vehicles utilize AC motors, DC is still used for power distribution due to certain efficiencies. The DC power is inverted to create AC current on board the vehicle. However, DC power has a tendency to produce stray current over unintended paths, particularly on the track side of the system. Therefore, stray current control is a necessary element in the design of the track system. Modern designs for DC transit systems include the concept of "source control" at the rail surface to minimize the generation of stray currents.

Electrical isolation of the rail using insulation is necessary for the protection of utility pipelines and steel structures along and near the LRT route.[2] In addition, if the track is shared by railroad freight traffic during non-revenue hours, insulated rail joints are required at all rail sidings and connections to adjacent rail facilities. The route of an LRT system is not generally within a totally dedicated right-of-way; therefore, the various types of track construction each require individual attention.

The essence of state-of-the-art technology in the design of modern transit systems is the concept of controlling stray current at the rails. Operation of the traction power system with the substation negatives isolated from ground (floating) will result in a higher overall system-to-earth resistance. The goal is to maximize the conductivity of the rail return system and the electrical isolation between the rails and their support systems.

The following are generally accepted design measures that the trackwork designer can use with various trackforms so as to create an electrically isolated rail system that controls stray currents at the source:

- Continuous welded rail.
- Rail bond jumpers at mechanical rail connections (especially special trackwork).
- Insulating pads and clips on concrete cross ties.
- Insulated rail fastening system for timber cross ties and switch timber.
- A minimum separation of 1 inch [25 millimeters] between the bottom of the rail and the ballast on ballasted track.
- Insulated direct fixation fasteners on concrete structures.
- Electrical insulation measures, such as enclosing the rail in a rubber boot or using a dielectric coating material at all roadway and pedestrian crossings.
- Special insulation materials for embedded rails, such as a rubber rail boot, an elastomeric grout, or surrounding the track slab in an insulating membrane.

- Cross bonding cables installed between the rails to maintain equal potentials of all rails and reduce resistance back to the substation.
- Insulation of the impedance bond tap connections from the housing case.
- Insulation of switch machines at the switch rods.
- Installation of rail-insulated joints to isolate rail-mounted bumping posts.
- Installation of insulated rail joints to isolate the main line from the yard and the yard from the usually grounded maintenance shop area.
- Separate traction power substations to supply operating currents for the main line, yard, and shop.
- Rail-insulated joints to isolate the main line rails from freight sidings or connections to other rail systems.

8.4.1 Rail Continuity

Continuous welded rail is the generally accepted standard for main line light rail construction. CWR creates an electrically continuous negative return path to the substation, in addition to other well-known benefits. There are, however, additional measures that can be taken to maximize the conductivity of the track system, thereby providing traction power less reason to stray.

8.4.1.1 Rail Joint Bonding

The rail configuration at special trackwork, turnouts, sharp curves, or crossovers may require jointed rails. Jumper cables, generally called “rail bonds,” permanently connected to the rail on either side of the bolted rail joint connections, ensure a continuous electrical path across the mechanical connections. Bond cables may be used to bypass complex special trackwork to provide continuity and also protect track maintenance workers from electrical shock when they are replacing special trackwork components. The use of jumpers must be carefully coordinated with the design of the signal system. See Chapter 10, Article 10.2.11 for additional discussion concerning rail bonding.

8.4.1.2 Cross Bonding

Periodic cross bonding of the rails and parallel tracks provides equivalent rail-to-earth potentials for all rails along the system. Using all parallel rails to return current provides a lower negative return resistance to the substation, since the return circuit consists of multiple paths rather than individual rails.

In “open” trackforms (e.g., ballasted or direct fixation track), where train control systems based on track circuits are employed, cross bonds are generally installed at impedance bond locations on rails to avoid interference with rail signal circuitry. Cross bonding is accomplished by attaching insulated cables to the rails. Both rails are connected in single-track locations, with all four rails cross bonded in double-track areas. The recommended method for cable connections to the rails will vary depending on the size of the cable. See Chapter 10, Article 10.2.11, for additional discussion on electrical bonding of rails for both traction power and signaling.

Cross bonding in embedded track sections requires an alternative design approach since the signaling system is not carried through the embedded track area. This is typically the case as most embedded track light rail systems run on "line-of-sight" operating rules coordinated with street traffic signal patterns.

To provide cross bonding of embedded tracks, insulated conduits are generally installed between rails prior to installation of the concrete for the initial track slab. Insulated cables are attached to each rail to obtain electrical continuity. These rail connections are typically enclosed in a covered box attached to the side of the rails so that the connections can be inspected and repaired if necessary. These boxes must be accounted for in the track slab design, including both storm water drainage and electrical isolation. Smaller cables may be used to provide an easier turning radius to the rails in the rail trough zone and facilitate attachment of the cables to the rails in constrained spaces. It is common design practice to install the cables at 1,000-foot [305-meter] intervals throughout the embedded track zone, with one such location being directly opposite each substation.

8.4.2 Ballasted Track Materials

The choice and details of materials used in track construction can have a significant effect on preventing stray current.

8.4.2.1 Concrete Cross Ties

Concrete cross ties with an insulated rail seat (generally consisting of a rail pad and clip insulators) provide good rail isolation. The rail seat pad is generally fabricated of thermo-plastic rubber, ethyl vinyl acetate, or natural rubber. It is approximately ¼ to $^{5}/_{8}$ inches [6 to 16 millimeters] thick and is formed to fit around the iron shoulders embedded in the concrete cross tie. The clip insulator is typically a glass-reinforced nylon material formed to sit between the rail base, the shoulder, and the elastic rail clip, electrically isolating each from the others. This assembly, plus a pad beneath the base of rail, electrically isolates the rail from the concrete tie. Insulating at the rail base is important because concrete cross ties, with their reinforcing steel, are not good insulators.

8.4.2.2 Timber Cross Ties

While wood is generally a good insulating material, timber cross ties are only marginal insulators because they are treated with preservative chemicals and they absorb moisture as they age. While timber cross ties provide sufficient insulation against low-voltage, low-amperage signal system currents, they offer little resistance to high-voltage, high-amperage traction power current.

Timber cross ties with insulating components at the fastening plate, as shown in Chapter 5 (Figure 5.4.2), can be used on main line track and at special trackwork turnouts and crossovers to reduce leakage. Electrical insulation can be achieved by inserting a polyethylene pad between the metal rail plate and the timber tie, installing an insulating collar thimble to electrically isolate the steel plate from the anchoring lag screw, and applying coal tar epoxy to the hole for the lag screw. The insulating pad and collar thimble afford insulation directly between the two materials. Coal tar epoxy applied to the drilled tie hole fills any void between the end of the collar thimble and timber tie and affords some insulation between the lag screw and wood tie. The insulated tie plate pad should extend a minimum of ½ inch [12 millimeters] beyond the tie plate edges to afford

a higher resistance path for surface tracking of stray currents. Chemical compatibility between the pad and epoxy material must be verified during design.

Timber cross ties can appear attractive compared to concrete ties on a first cost basis; however, the cost of materials and labor associated with adding insulated rail fastenings to timber cross ties can tip the scale toward the use of concrete cross ties. This is particularly true if high-quality timber ties are specified. Because of this factor, the longer life expectancy of high-quality concrete cross ties, and the fact that fewer concrete cross ties are usually required, most rail transit projects have concluded that concrete cross ties are more economical on a life cycle cost basis.

8.4.2.3 Ballast

To eliminate the path for stray current leakage from rail to ballast, the ballast section should be a minimum of 1 inch [25 millimeters] below the bottom of the rails. The clearance requirement pertains to rail on both concrete and timber cross ties for both main line and yard trackage. This is essential to increase the rail-to-earth resistance and assist in minimizing the stray current leakage to earth. Ballast should be clean and well-drained not only when initially constructed, but also during service. Regular maintenance is required, as described in Chapter 14 of this Handbook. The use of metallic slags as ballast is not recommended. Rail grinding should be done with vacuum systems to minimize contaminating ballast with metallic grindings.

8.4.3 Embedded Track Issues

Embedded track is often used on portions of a light rail system where the tracks are located within an urban street. Electrical isolation of embedded rails can be provided by insulating all the surfaces of the rail with the exception of those in contact with the wheels, insulating the trough that the rail sits in, or a combination of both. Track may also be isolated by insulating the perimeter of the entire concrete base slab, a concept often called the “bathtub” stray current isolation concept. The materials used to provide this insulation generally consist of polyethylene sheeting, epoxy coal tar coating, elastomeric grout, or natural rubber sheeting. Rail boot, first used on Toronto’s streetcar system in the 1990s, has very nearly become the default method of electrically isolating plain embedded track, but several insulating systems have been used successfully. The specific design for stray current control is selected by the track designer with recommendations from the corrosion control specialists. See Chapter 5 for additional information concerning details and materials for insulation of embedded track.

8.4.4 Direct Fixation Track Issues

Direct fixation track is generally located on aerial sections or in tunnels in light rail transit systems. The direct fixation fasteners, as described in Chapter 5, provide electrical insulation between the rails and the concrete structure. The typical direct fixation fastener consists of a sandwich of steel plates and rubber pads, with the latter providing both electrical isolation and acoustic attenuation. An elastomer of the proper resistivity provides excellent insulation and deters current leakage. Fastener inserts are often epoxy coated to further isolate the rails from the concrete slab.

Despite the use of insulated rail fasteners, stray current leakage often occurs in damp tunnels that are subject to ground water seepage. The fasteners can become coated with a wet conductive film. The problem can be particularly acute in snowbelt climates, as the entire tunnel track system can become coated with a brine composed of de-icing chemicals mixed with snow that is carried into the tunnel from surface portions of the system. At stations, even those outside of tunnels, the area below the rail and between direct fixation fasteners will often become filled with a mixture of water, brine, grease and the sand that is dispensed by the LRVs to improve traction when braking—creating a direct short to ground. A regular maintenance program is essential to keep the track structure clean in tunnels, since natural precipitation is not available to wash the track in these areas. The importance of routine housekeeping to flush these contaminants off the track cannot be overstated. See Chapter 14 on LRT track maintenance for additional discussion on this topic.

8.4.5 Yard and Shop Track Issues

Maintenance shop tracks are grounded to protect workers by reducing the touch potential between the rail car and ground to zero. Maintenance yard tracks are generally floating or non-grounded, but insulation is rarely included between the rails and the timber cross ties. This design decision is based on economic considerations, as well as the fact that yard tracks are generally used only at low speed and consequent low current draw, and a separate traction power substation is used to supply operating current for train movement in the yard. The only time the yard rails become electrically connected to the main line or shop rails is when a train enters or leaves the yard or shop. This is a short period and does not result in any harmful sustained current leaking into the earth.

Note that transit system structures within a yard complex may have to be protected against locally originated stray currents between yard trackage and the yard substation. Consequently, underground utilities in yards are often constructed with non-metallic materials such as PVC, FRE, and polyethylene.

8.4.6 Track Appliances

Several items that are attached to the track can circumvent barriers to stray current if not detailed correctly. Many of these are related to the LRT train control system. The paragraphs below discuss some of the devices that can require special attention so as to not become stray current leakage locations.

8.4.6.1 Impedance Bonds

Leakage of stray currents into the earth can be a significant problem if the cables from the rails are electrically connected to an impedance bond housing case that is in contact with the earth. This type of grounded installation can result in a continuous maintenance problem if an effectively high rail-to-earth resistance is to be achieved. Instead, the housing case should be mounted clear of any concrete slab conduits, reinforcing bar, and contact with the earth.

Impedance bond housing cases for a light rail transit line are generally located at-grade along the right-of-way. The cases are mounted on timber tie supports in the ballasted area either between or directly adjacent to the rails. In order to eliminate possible points of contact with the earth, the

center taps of the impedance bonds are insulated from the mounting case by installing a clear adhesive silicone sealant between the center taps and the case.

While the track engineer is generally not involved in the placement of impedance bonds, he or she should examine the details proposed by the signal and traction power design staff so as to be certain the installation provides proper isolation. On more than one occasion, the track design has been blamed for a stray current leak that turned out to be the fault of a deficient impedance bond installation detail. See Chapter 10, Article 10.2.2, for additional discussion concerning impedance bonds.

8.4.6.2 Switch Machines

For several reasons, including the safety of the signal maintainers, electrically powered switch machines need to be grounded. This requires that they be electrically insulated from the track structure. See Chapter 10 for discussions concerning switch machines.

8.4.6.3 End-of-Track Bumping Posts and Buffers

It has been common practice to electrically isolate rail transit end-of-track devices from the track by installing insulated rail joints in the track ahead of the striking face of the unit. This is not actually necessary for all such installations. There are several issues involved:

- Making certain the end-of-track device does not create a shunt between the rails, thereby giving the signal system (if present) a false indication of track occupancy.
- Making certain that the end-of-track device does not become a stray current leak.
- Making certain the working parts of a hydraulic buffer end-of-track device are protected from corrosion.

Static fixed bumpers with no moving parts and friction buffers that control train deceleration solely with rail skates and/or spring-loaded heads require only a single insulated joint in one rail to interrupt signal shunt current. So long as the track they are sitting upon is isolated from the earth with the ordinary details for that trackform, full isolation of such devices from the running rails is not necessary. The practice of two insulated joints is likely a holdover from the days when ballast piles over the rails were used as an end-of-track control device. Since the ballast pile grounded the rails, the insulated joints were necessary.

End-of-track buffers with hydraulic heads may require grounding to protect the hydraulic working parts from corrosion. Since the grounding system would create a stray current path, those types of end-of-track devices should have an insulated joint in both rails.

8.4.7 Stray Current Tests and Procedures

The track isolation methods discussed in the articles above, when properly constructed, will provide good initial values of rail-to-earth resistance. However, as these components deteriorate, they become dirty and require maintenance to maintain their original resistivity. Periodic tests are also required to locate and remove direct shorts that occasionally occur as discussed in the

following section. Stray currents can rise to harmful levels if short circuits to ground are not detected and removed.

Once the LRT system is in operation, regularly scheduled tests are recommended to monitor and maintain the integrity of stray current control systems. The most common tests are rail-to-earth resistance tests, substation-to-earth voltage tests, and structure-to-earth tests. Transit agencies use a broad spectrum of approaches for these test programs ranging from infrequent use of consultants to permanent in-house corrosion control personnel. Typically, the greatest efforts are made only after stray current problems have already damaged piping, utility structures, trackwork components, or signal circuits. Such troubleshooting can resolve problems, but it is much more effective to conduct regularly scheduled, routine monitoring for stray currents so that problems can be detected and corrected before they manifest themselves in the form of measurable corrosion or degraded signal system performance.

Track resistance to ground is a major indicator of the isolation of the electric traction return system from the infrastructure. The generally recognized methodology for the determination of track resistance is given in ASTM G165 – 99 Standard Practice for Determining Rail-to-Earth Resistance. Sections of track should be tested as they are constructed rather than only upon the completion of construction in order to immediately identify and repair a failed section. See Chapter 13 for additional discussion on this topic.

8.5 SUMMARY

Corrosion from stray electrical currents is an important interdisciplinary issue that requires the close attention of the design team. There are several effective methods that have been used by light rail system operators and builders to either avoid or mitigate the effects of stray current corrosion. The designer must seek the advice of experts in this complex field, as well as coordinate with utility companies and the signal system designer. It is also important to recognize that track component specifications should include appropriate electrical resistance features to accomplish the goals of the corrosion control plan. However, the designer needs to be certain that specified products will actually meet the specified performance requirements for track-to-ground resistance of the entire track system.

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Chapter 9—Noise and Vibration Control

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CHAPTER 9—NOISE AND VIBRATION CONTROL

9.1 INTRODUCTION

Wayside noise and vibration and car interior noise are important factors in the design of new transit track or retrofit of existing track. All too often, noise and vibration are ignored until well into the design phase, at which point incorporation of the most cost-effective solutions may not be possible. Successful noise and vibration control require consideration of both the track and the vehicle as a system, because the interaction of the wheel and the rail is responsible for the bulk of wayside noise and vibration impacts.

This chapter attempts to summarize the main issues and theories related to noise and vibration production and control and the related issues concerning rail wear, identify areas where insufficient knowledge exists, and provide guidance with respect to track design for acceptable levels of noise and vibration. While many of the treatments considered here can be selected and designed by the transit track engineer, the design of some provisions, such as floating slabs or vibration absorbers, should be conducted by those who have considerable experience with designing and specifying vibration isolation systems, have an engineering or physics background, and understand the principles of noise and vibration control.

Noise and vibration control provisions should be included in track design to avoid impacting wayside communities, transit users, and transit employees. Noise and vibration can usually be held to acceptable levels at reasonable cost with appropriate design and maintenance provisions, especially if the vehicle and track are considered as a system rather than as separate, independent components. For example, expensive track vibration isolation systems might be avoided where vehicles with low primary suspension vertical stiffness are used, whereas vehicles with high primary suspension stiffness might produce vibration that might require a floating slab to isolate the track—an expensive proposition. The choice of vibration isolation provisions depends on vehicle dynamic characteristics, and the track and vehicle design teams must coordinate their designs during and after the early stages of any project. Mitigation could involve considerable expense, weight, space, or special procurements. Late consideration of noise and vibration isolation may preclude some treatments simply because insufficient time exists to obtain them or to implement design changes.

9.1.1 Scope

This chapter is divided into three principal topics, the first being wheel/rail noise, the second structure-borne and ground-borne noise and vibration, and the third wheel/rail contact stresses. All of these topics are interrelated. For example, fastener selection for ground- or structure-borne noise control may affect wheel/rail noise, and rail corrugation is influenced by track design parameters and contact stresses.

Vehicle "on-board" treatments are not discussed here, as these are beyond the limits of track design. However, considerable discussion is included regarding vehicle design parameters that may influence the selection of track design provisions. Low-profile sound barriers located close to the rail or acoustical absorption placed between the rails may be considered as part of the

track design, as they may influence the track support design and maintenance. However, wayside sound barriers would not be considered under track design.

9.1.2 Relevant Literature

Many studies of rail transportation noise and vibration have been conducted, producing detailed technical reports containing comprehensive information concerning rail transit noise and vibration prediction and control. Particularly useful sources of information include the following:

- The FTA Guidance Manual, Transit Noise and Vibration Impact Assessment and Control[1]
- The *Handbook of Urban Rail Noise and Vibration Control*, a handbook on all aspects of rail transit noise and vibration control published by US DOT/TSC (now the John A. Volpe National Transportation Systems Center),[2] a document that should serve as a companion to this chapter.
- A review of the state-of-the-art in wheel/rail noise control has been prepared in *TCRP Report 23*, which includes numerous references to technical reports and other literature, and which provides guidelines for selection of rail transit noise control provisions.[3]
- A review of ground-borne noise and vibration prediction and control was performed in 1984, including preparation of an annotated bibliography.[4] The document provides information concerning vibration isolation performance for various track design configurations.
- The proceedings of the International Workshop on Railway and Tracked Transit System Noise (IWRN), which are usually published in the *Journal of Sound and Vibration*.[5] The proceedings have recently been published by Springer.[6]

Many papers concerning noise and vibration prediction and control are published each year in TCRP Research Results Digests, and journals such as the *Journal of Sound and Vibration*, *Wear*, *Journal of the Acoustical Society of America*, *Transportation Research Record*, and so on. General handbooks concerning noise and vibration control and design include the *Handbook of Noise Control*[7] and the *Shock and Vibration Handbook*.[8]

9.1.3 Some Fundamentals

Excellent references to acoustical and vibration terminology are provided by Beranek[9], Harris[10], and by Harris and Crede.[11] Also, the usage employed here is consistent with the FTA guidance manual.[13]

9.1.3.1 Sound Levels

Sound is a dynamically varying pressure fluctuation in air that occurs over the frequency range of human hearing, here taken to extend from about 16 Hz to 20 KHz. Sound is conveniently described with a logarithmic decibel scale (dB). Mathematically, the level in decibels of a sound is equal to 20 times the logarithm to base 10 of the ratio of the root-mean-square sound pressure and a reference pressure of 20 micro-Pascal:

$$L(dB) = 20\ \mathrm{Log}_{10}\ (p/p_0),\ \ p_0 = 20 \times 10^{-6}\ \mathrm{Pa}$$

$$= 10\ \mathrm{Log}_{10}(p^2/p_0^{\,2})$$

The A-Weighted sound level is obtained by filtering the analog sound pressure signal obtained with a pressure-sensitive microphone with an A-Weighting network that deemphasizes the low-

frequency components of noise below about 500 Hz and above about 10 KHz.[12] The A-Weighting network approximates the frequency response of a person's ear at low listening levels and is used almost universally to characterize industrial, occupational, and community noise. The unit of the A-Weighted noise level in decibels is abbreviated as "dBA." Unless otherwise indicated, all sound levels discussed here are in dBA.

The energy associated with a sound pressure is proportional to the square of the sound pressure amplitude. The above formula indicates that reducing the sound energy by a factor of 2 reduces the sound level by only 3 decibels, a difference that may be barely perceptible to the casual listener if frequency and event characteristics remain unchanged. An increase or decrease of sound level by 10 dB is often interpreted as a doubling or halving, respectively, of perceived, sound, even though the sound energy increases or decreases by a factor of 10.

9.1.3.2 Vibration Levels

Vibration is commonly described in terms of displacement, velocity, or acceleration. Velocity correlates well with human response, and criteria for human exposure to vibration produced by rail transit operations are stated in terms of vibration velocity level in decibels. The decibel scale represents vibration in the same manner as the does the decibel scale for noise. The unit of vibration velocity level is designated as VdB, dBV, or simply dB. Unless otherwise stated, vibration velocity levels will be in decibels relative to 1 micro-inch per second:

$$L(VdB) = 20\ Log_{10}\ (v/v_0),\ v_0 = 1 \times 10^{-6}\ in/sec$$

The metric system is used around the world, notwithstanding the engineering community in the United States. Reference vibration velocity magnitudes used for defining the decibel scale include 10^{-6} m/sec (1 micron per second), 10^{-6} cm/sec, 5×10^{-6} cm/sec, and 10^{-9} m/sec.

Vibration acceleration is also described with a decibel scale. Reference acceleration magnitudes include the micro-g (9.8×10^{-6} m/sec) and 10^{-6} m/sec^2. The reference velocity and acceleration employed here are 1 micro-in/sec (25 micron/sec) and 1 micro-g (9.8×10^{-6} m/sec).

The ground-borne or structure-borne sound produced by a vibrating surface such as a building wall, wheel, or rail is directly proportional the vibration amplitude of the surface at specific frequency. Thus, a 10-decibel change in the level of a vibrating surface produces the same 10-decibel change in radiated sound, greatly simplifying the calculation of sound levels from vibration levels. Further, the attenuation of vibration or noise due to various vibration and noise control provisions is most conveniently described with decibels. Thus, if the sound level is attenuated by 10 dB from point A to point B, the level at point B is simply determined by subtracting 10 dB from the sound level at point A.

9.1.4 Design Criteria

Guidelines have been developed by the Federal Transit Administration (FTA)[13] and the American Public Transportation Association (APTA)[14]. The FTA guidance manual provides criteria for both airborne noise and vibration in terms of the energy equivalent levels (L_{eq}) and day-night levels (L_{dn}) and ground-borne noise in terms of maximum levels and integrates the noise impact analysis for combined rail and bus transit operations. The FTA guidance manual

recommends criteria for floor vibration in residences and institutions. The residential vibration criteria follow the recommendations provided in ANSI Standard S2.71-1983 (R 2006).[15] These criteria are for maximum ⅓ octave band root-mean-square vibration velocity levels measured over any 1-second period during vehicle passage. The FTA guidance manual is generally used to assess impacts for federally funded projects and is recommended by the FTA for all rail transit projects, including bus transit. Refer to the FTA guidance manual for detailed description of the recommendations.

The APTA guidelines recommend limits on maximum pass-by noise levels (i.e., the maximum noise levels that occur during an individual vehicle or train pass-by), as well as limits on the noise caused by ancillary facilities (i.e., fixed services associated with the transit system). The APTA guidelines have been supplanted by the FTA guidelines and are rarely, if ever, used for new construction. For most practical situations, the wayside noise levels and impact mitigation measures resulting from applying the FTA and APTA guidelines are very similar, although not identical. Design criteria for many existing transit systems were based on the APTA guide. No criteria for ground vibration are provided in the APTA guide. TCRP Project D-12 developed criteria for acceptable levels of rail-transit-generated, ground-borne noise and vibration in buildings. See *TCRP Web-Only Document 48* for more details.

9.2 NOISE CONTROL DESIGN

This article addresses wheel/rail noise and track-based noise control treatments. This very complicated subject has received the attention of numerous researchers and practitioners, and a chapter such as this cannot address all of the aspects of wheel/rail noise in complete detail. Even so, sufficient detail is provided to expose the user to various aspects of wheel/rail noise and provide direction for further investigation as may be desired.

Article 9.2.1 addresses the rolling noise that occurs on tangent track or moderately curved track. This is further broken down into “normal rolling noise”, “impact noise”, rail corrugation noise, grinding artifact noise, and singing rail. Article 9.2.2 deals with special trackwork noise, which is a form of impact noise related to special trackwork. Article 9.2.3 is concerned with curving noise, including wheel squeal and flanging noise, one of the most significant types of rail transit noise.

Transit vehicle auxiliary equipment also produces noise. Examples include roof-mounted HVAC equipment, brakes, and horns. These vehicle auxiliary equipment noise sources are not specifically treated here, although they may influence wayside-based noise control provisions. The user is referred to *TCRP Report 23, Wheel/Rail Noise Control Manual*. The material presented below is focused primarily on track-based noise control design

9.2.1 Wheel/Rail Rolling Noise

Rolling noise is produced primarily by rail and wheel surface roughness with characteristic wavelength of several inches down to a fraction of an inch. Rolling noise is distinct from curving noise, both in nature and in generating mechanism. Rolling noise is radiated by the wheels and rails and may also be radiated by the structure supporting the track, such as elevated steel or concrete structures. This latter form of structure-radiated noise is termed “structure-borne” and is dealt with in the section regarding structure-borne and ground-borne noise and vibration control.

9.2.1.1 Types of Rolling Noise

The categories of wheel/rail noise include the following:

- Normal rolling noise
- Impact noise due to loss of contact between the wheel and rail, caused by rail head defects, gaps, and joints
- Rail corrugation noise, sometimes referred to as roaring rail noise
- Grinding artifact noise
- Singing rail

9.2.1.1.1 Normal Rolling Noise

Normal rolling noise is broadband noise produced by reasonably smooth rail and wheel treads. The following generating mechanisms have been identified as sources of normal rolling noise:

- Wheel and rail roughness
- Parameter variation of rail head geometry or moduli
- Dynamic creep
- Aerodynamic noise

Wheel and Rail Roughness

Wheel and rail surface roughness is the major cause of rolling noise; the greater the roughness amplitude, the greater the rail vibration and wayside noise. Assuming that the wheel/rail contact stiffness is infinite, the rail and wheel tire would displace relative to each other by an amount equal to the sum of their roughness amplitudes. The ratio of rail motion relative to wheel motion at a specific frequency will depend on the dynamic characteristics of the rail and wheel and the contact stiffness, all of which are represented by their respective mechanical impedances as a function of frequency.[16]

At short wavelengths relative to the contact patch dimension, surface roughness is believed to be attenuated by averaging the roughness across the contact patch in a direction parallel with the rail, an effect known as contact patch filtering.[17] Thus, fine regular grinding or milling marks less than 0.040 to 0.80 inch [1 or 2 mm] in wavelength would produce less noise than roughness of equivalent amplitude at wavelengths on the order of an inch.

Excessive wheel/rail conformity has been identified as a cause of spin creep corrugation, leading to increased noise.[18] Secondly, the contact mechanical impedance (stiffness) between the wheel and rail becomes large with a high degree of conformal contact, and may exceed the mechanical impedances of the rail and of the wheel tread, causing high-frequency contact forces. Good low-noise performance has been obtained at the BART ballast-and-tie test track with 119RE rail (modified through rail grinding to about a 10-inch [300 mm] crown radius) and cylindrical wheels that provide a rounded contact geometry, indicating that minimal conformal contact is desirable for low-noise operation on tangent track.[19] High conformal contact may produce lower contact stresses in the rail head than a rounded contact zone for the same force, but this benefit may be canceled by high-frequency contact forces and spin slip. More discussion of this is provided below.

Parameter Variation

Parameter variation refers to the variation of contact stiffness due to variation of the rail head crown radius and variation of rail and wheel steel elastic moduli.[17] The effects of fractional changes in Young's elastic modulus and of the radius of curvature of the rail head as a function of wavelength necessary to generate wheel/rail noise equivalent to that generated by surface roughness are illustrated in **Figure 9.2.1**. The wavelength of greatest interest is 1 to 2 inches, corresponding to a frequency of about 500 to 1,000 Hz for a vehicle speed of about 55 mph, the maximum speed of typical light rail vehicles. Over this range, a variation in modulus of 3 to 10% is required to produce the same noise as that produced by normal rail roughness.

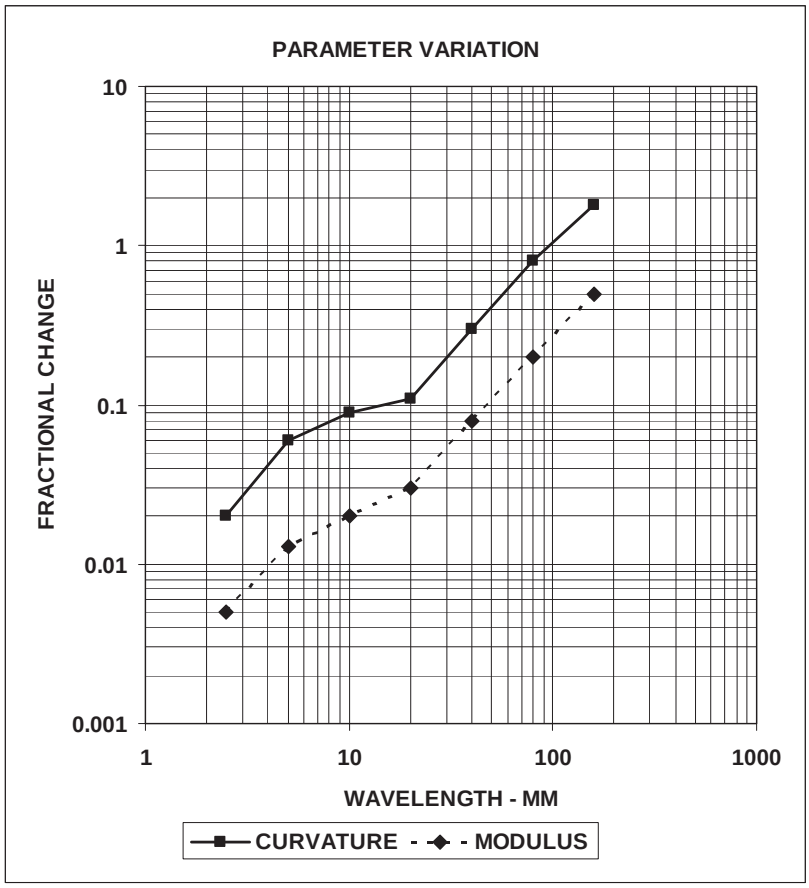

Figure 9.2.1 Change in elastic modulus and rail head curvature required to generate wheel/rail excitation equivalent to roughness excitation (Remington, 1988)[17]

Variation of rail head ball or crown radius of curvature also induces a dynamic response in the wheel and rail. A variation of rail head curvature on the order of 10 to 50% produces noise levels similar to those produced by rail height variation alone. Rail head crown radius variation will normally accompany rail height variation, especially where corrugation is present or a high degree of conformal contact exists. Maintaining a uniform rail head radius is necessary to fully realize the benefits of grinding rail to maintain uniform head height. Irregular definition of the contact wear strip indicates excessive rail head radius variation. Reducing the width of the contact zone, thus reducing conformal contact, also reduces the sensitivity to head curvature variation. A high degree of conformal contact exacerbates the effects of rail head radius variation.

Dynamic Creep

Dynamic creep includes both longitudinal and lateral dynamic creep, roll slip parallel with the rail, and spin creep of the wheel about a vertical axis normal to the wheel/rail contact area.

Longitudinal creep is wheel creep in a direction parallel with the rail and is usually not considered in design and noise analysis. However, qualitative changes in wheel/rail noise on newly ground rail with an irregular transverse grinding pattern in the rail surface are audible as a train accelerates or decelerates, suggesting that longitudinal creep may play a role. Secondly, one may observe longitudinal striations in rail corrugation with martensite formation, presumably caused by frictional heating of the contact area.

Lateral creep is wheel slip across the rail running surface in a direction transverse to the rail, and is the cause of wheel squeal. Lateral creep at tangent track may occur during unloading cycles at high frequencies on abnormally rough or corrugated rail and may be responsible for short-pitch corrugation at some tangent track. Lateral creep may occur at tangent track due to poor truck steering (crabbing) caused by poorly matched wheel diameters and wheel tapers. Extreme crabbing may be caused by false flanging, where the field side of the tire tread has a larger radius than the middle running surface of the tire tread. The cure for this latter condition involves regular wheel truing, as opposed to track maintenance. Cylindrical wheels do not promote crabbing unless their diameters are mismatched within a wheel set, but also do not promote steering or prevent crabbing. Moreover, a cylindrical wheel profile may rapidly develop a negative taper (false flange). Some degree of wheel taper is desirable to increase the time before a false flange can develop prior to truing. In general, a false flange condition should never be allowed to develop.

Spin creep is caused by a wheel taper that produces a rolling radius differential between the field and gauge sides of the contact patch. Spin rotation of the wheel tire is rotation about a vertical axis through the tire center. Spin creep has been associated with high corrugation rates.[18]

Roll slip refers to rolling contact with slip at the edges of the contact zone. Some slip, continuous or otherwise, is required at the edges of the contact zone, as with Heathcote slip of a bearing in its groove, required by the conformal contact of curved contact surfaces. Reduction of the contact zone width would reduce conformal contact and thus potentially reduce roll slip. However, roll slip is a necessary consequence of the finite size of the contact zone and cannot be eliminated. Wheel and rail vibration modes may influence the formation of corrugation due to roll slip.

Aerodynamic Noise

Aerodynamic noise, due to high-velocity air jets emanating from grinding grooves in the rail, has been claimed to produce a high-frequency whistling noise.[20] No test data have been obtained to confirm this claim. This noise, if present, may be indistinguishable from wheel/rail noise radiation. Fine rail grinding (acoustic grinding) that avoids course grinding marks should, presumably, reduce this type of noise, if it exists.

Other sources of aerodynamic noise include air turbulence about the wheels and trucks and traction motor blower noise. Neither of these is controllable by the track designer, but traction

motor blower noise can, under certain circumstances, dominate the wayside noise spectrum if not properly treated. Aerodynamic noise due to air turbulence about the wheels and trucks at light rail transit speeds should not be significant. Ballasted track will tend to reduce aerodynamic and auxiliary equipment noise by absorbing sound beneath the vehicle.

9.2.1.1.2 Impact Noise

Impact noise is a special type of wheel/rail noise occurring on tangent track with high amplitude roughness due to special trackwork, corrugation or spalling, rail joints, rail defects, or other discontinuities in the rail running surface and wheel flats. Impact noise is probably the most apparent noise on older transit systems that do not practice regular rail grinding and wheel truing. Even with continuous welded rail, rail welds and insulating joints must be carefully formed to reduce impact noise generation. Rail joint maintenance is important on older systems employing jointed rail.

Remington[21] provides a summary of impact noise generation that involves non-linear wheel/rail interaction due to contact separation and is closely related to impact noise generation theory at special trackwork. I. L. Ver categorizes impact noise by type of rail irregularity, train direction, and speed.[22] Interestingly, impact noise becomes less dominant relative to general rolling noise as speed increases. This occurs for spalls in the rail head or rail gaps, for example, where the wheel actually traverses the gap before the rail can rise up to the wheel.

9.2.1.1.3 Rail Corrugation Noise

Rail corrugation is periodic rail roughness with longitudinal wavelengths extending from a fraction of an inch up to 6 inches [1 cm up to 15 cm] or more. Short-pitch corrugation is most relevant to wayside noise from rail transit systems. Rail corrugation may be of low amplitude, as during its initial stages, or may involve deep corrugation leading to wheel/rail contact separation (impact noise). Rail corrugation with contact separation causes excessive rolling noise of a particularly harsh character and very high sound level. The terms "roaring rail," “roar,” and "howl" describe noise produced by corrugated rail. If rail corrugation exists, the wayside noise level will be higher than that of normal rolling noise by as much as 10 decibels, and the frequency spectrum will contain peaks and associated harmonics, producing an unpleasant tonal character.

Examples of wayside noise from corrugated and uncorrugated rail are presented in **Figure 9.2.2**. These data are ⅓ octave band plots of noise measured at about 50 ft [15 m] from ballast and tie track. All of the vehicles represented had resilient Bochum 54 wheels. The data collected at Sacramento RTD and the Los Angeles Blue Line contain a pronounced peak at 800 Hz, which is due to moderate rail corrugation at about 1.5 to 2 years after the last grinding and possibly grinding marks.[23] The data shown for the Portland TriMet system were obtained about 5 years after the last grinding, but do not contain a peak at 800 Hz. An inspection of the wheels at the Sacramento RTD and Los Angeles Blue Line indicated moderate wheel tread hollowing with moderate false flange. The Portland TriMet wheels are ground regularly, and inspections of these wheels suggested little false flange. Whether or not wheel truing practices contribute to rail corrugation at the Sacramento RTD or the LAMTA Long Beach Blue line is a matter for additional investigation, and conditions may have changed considerably since these data were collected. The tie spacing at these tracks is nominally 30 inches [750 mm], which supports a pinned-pinned mode in the rail at about 800 Hz for RE115 rail, which may contribute to rail corrugation growth.

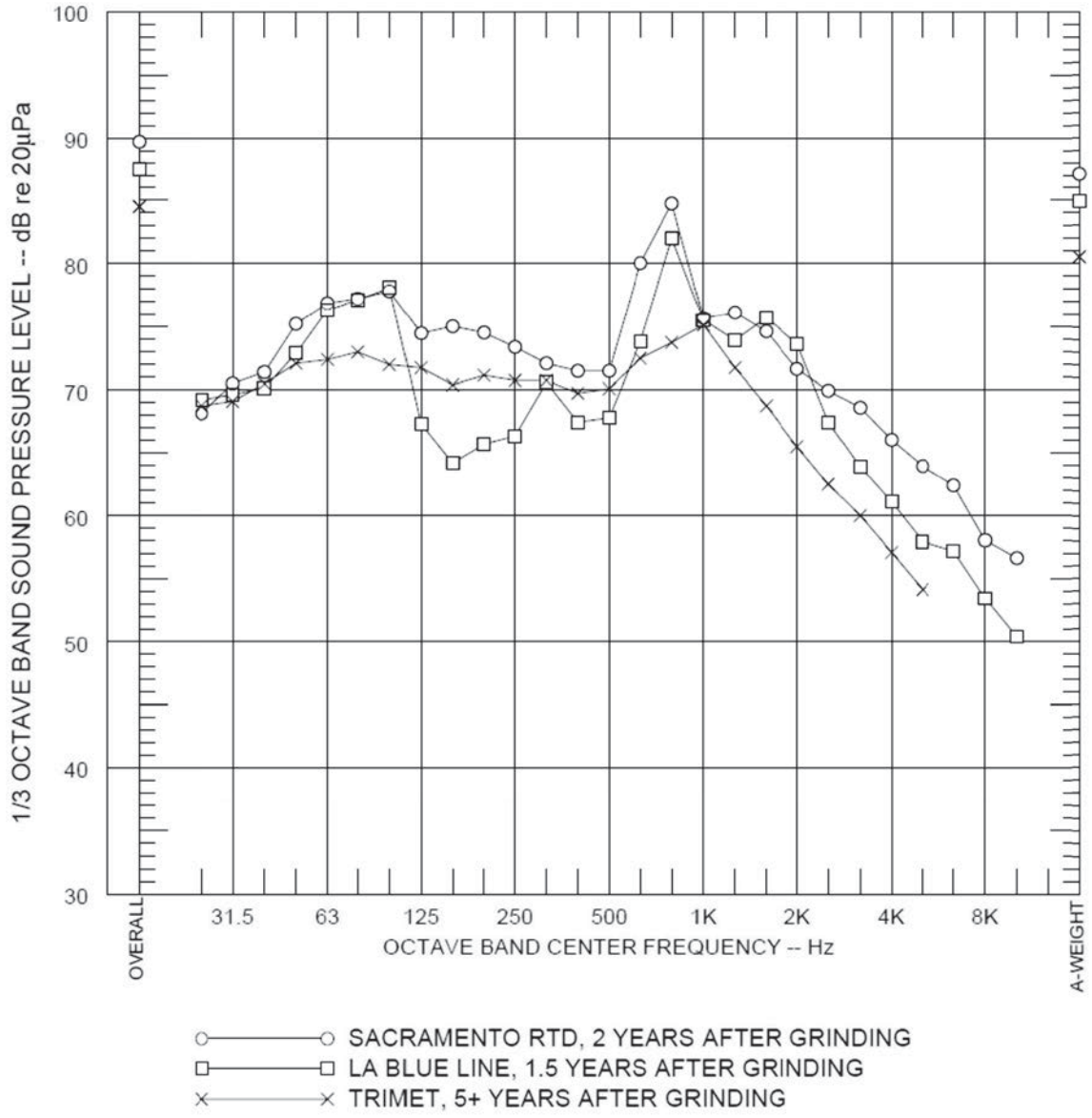

Figure 9.2.2 Examples of wheel/rail noise

Rail corrugation is more difficult to control on rail transit systems than railroads because of the lower contact static loads, the uniformity of transit vehicles, and the uniformity of speeds that prevent randomization of wheel/rail forces. Low contact loads allow the wheel and rail to slip more readily than do high contact loads, allowing wheel/rail contact separation to occur on rail with high amplitude corrugation. Wheel/rail contact separation may involve electrical arcing where high currents are required for acceleration and regenerative braking of the vehicle. Thus, maintaining rail smoothness is probably more important for transit systems than heavy freight systems. Rail corrugation is the principal cause of excessive noise levels on many transit systems, and controlling rail corrugation is key to minimizing rail transit system noise. At present, the most effective means of controlling rail corrugation is rail grinding. Detailed discussions of rail

corrugation noise are included in *TCRP Report 23*.[3] Rail grinding, including acoustic grinding, is discussed in detail by Zarembski.[24]

Figure 9.2.3 is a photograph of very short-pitch corrugation for a light rail system with embedded track and resilient wheels. The wavelength is less than 1 inch [2.5 cm], and the width is relatively small compared to the width of the rail head. The lighter portions of the periodic patterns are referred to as "white etching layer" in the literature. Wild et al. indicates that the white etching layer is due to nanocrystalline martensite and cementite particles. The white etching layer occurs at the peaks of the corrugation, where the steel is generally much harder than at the troughs.[25] According to Hutchings, martensite is formed by heating and rapid quenching by conduction of heat into the bulk of the metal and may also be facilitated by shear stress.[26] Both of these processes may occur during wheel/rail contact dynamic slip, the heating due to friction. The longitudinal striations suggest a longitudinal slip, perhaps related to torsional oscillation of the wheel set. There is no evidence of rail grinding marks that might have contributed to this pattern.

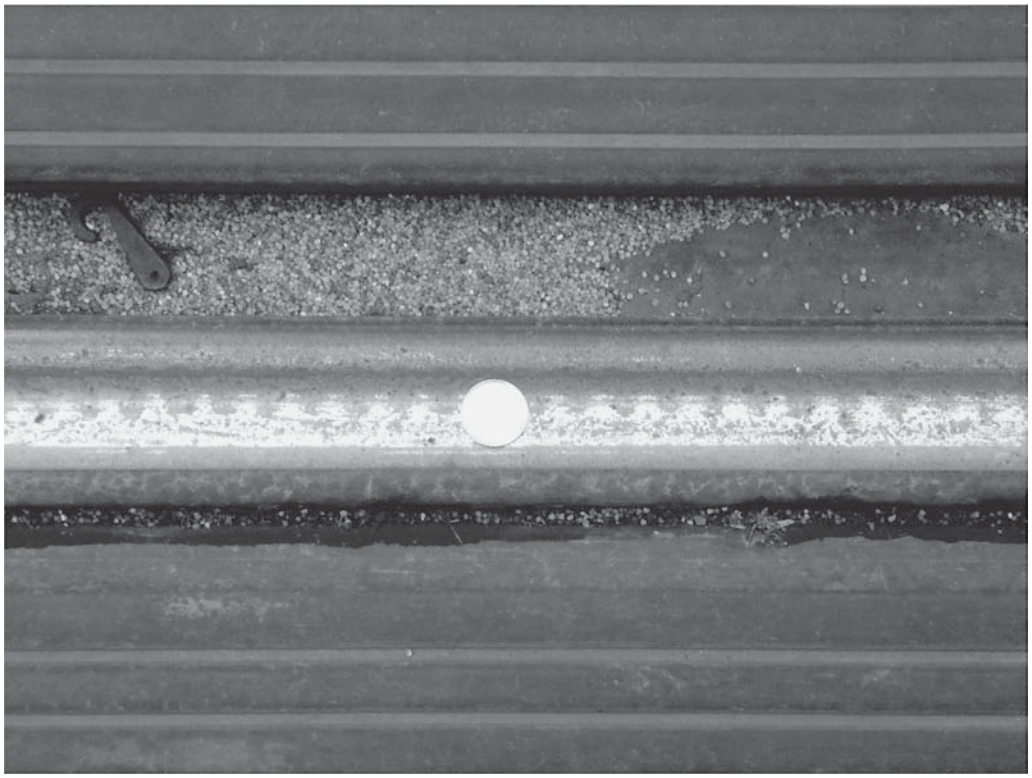

Figure 9.2.3 Very short pitch corrugation

Very short-pitch corrugation has been claimed to occur at Sound Transit embedded and direct fixation track. The nature of the corrugation appears to be related to rail grinding marks, but a dynamic mechanism may also be responsible. (This is discussed in more detail regarding rail grinding artifact noise in Article 9.2.1.1.4 of this chapter.)

Figure 9.2.4 is a photograph of conventional short-pitch corrugation of wavelength on the order of 3 to 4 inches. This was observed a few feet away from the very short pitch corrugation shown in

Figure 9.2.3. Again, martensite formation and longitudinal striations suggest longitudinal slip. These corrugations were observed at a relatively short section of track, where three-car trains accelerated from a station stop through the corrugated section. Thus, train speed was varying, in spite of the apparent uniform wavelength. The maximum train speed was probably about 15 to 20 mph. The vehicles were low-floor vehicles with resilient wheels. The rail was evidently 115 RE, embedded in an elastomer. The corrugation appeared to be an anomalous condition at this system, but is representative of corrugations at many other systems. In particular, the uniformity of wavelength in the presence of varying train speed suggests a geometric cause of the corrugation, as opposed to a mechanical resonance. The geometric parameters involved include rail head ball radius, and wheel tread profile and radius. The nature of the rail support was not determined. These corrugations produced a low-frequency rumble (ground-borne noise) that was perceptible to the feet. However, the corrugation depth was not perceptible to touch.

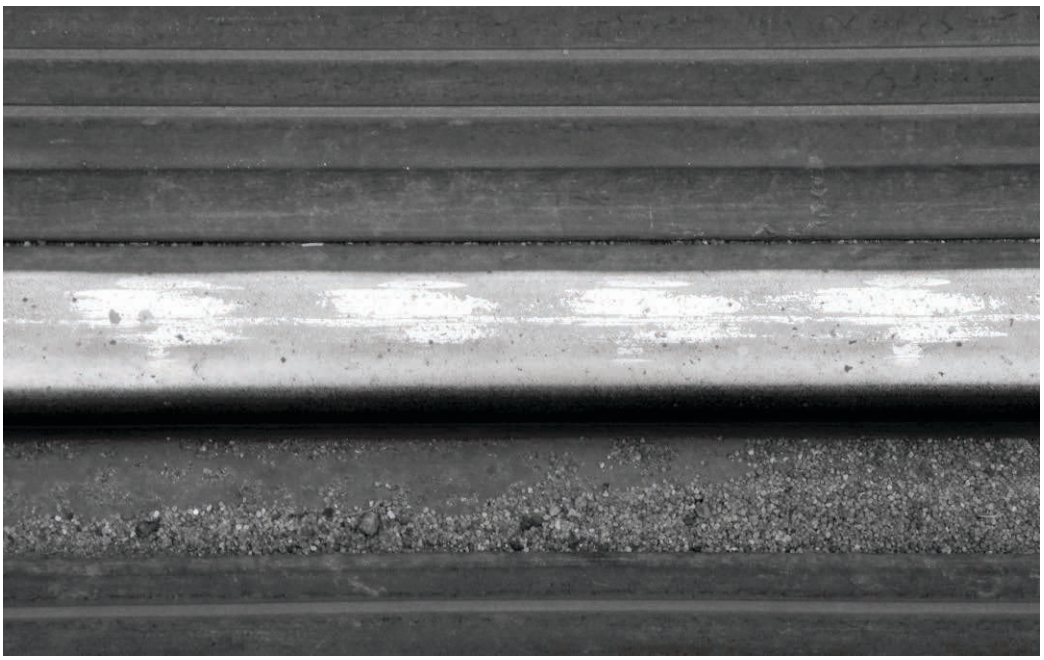

Figure 9.2.4 Short-pitch corrugation

Figure 9.2.5 is a photograph of rail corrugation on the California Street cable car line in San Francisco. The cable car wheels are of small diameter and are non-powered since the vehicles are towed by the cable hidden beneath the street surface. The corrugation patterns extend across most of the rail head and are very regular, with a wavelength of less than 1 inch. These corrugations do not occur on the downhill slopes, possibly because of wood block brakes that rub the rail to help control descent (on the cable.) One possibility is that the resins left by the blocks may affect wheel/rail contact adhesion; another is that the blocks simply smooth out the corrugation before it can occur. While the cable car system is not representative of typical light rail systems, the corrugation processes may be very similar, and attention is called to the high degree of wheel rail conformal contact.

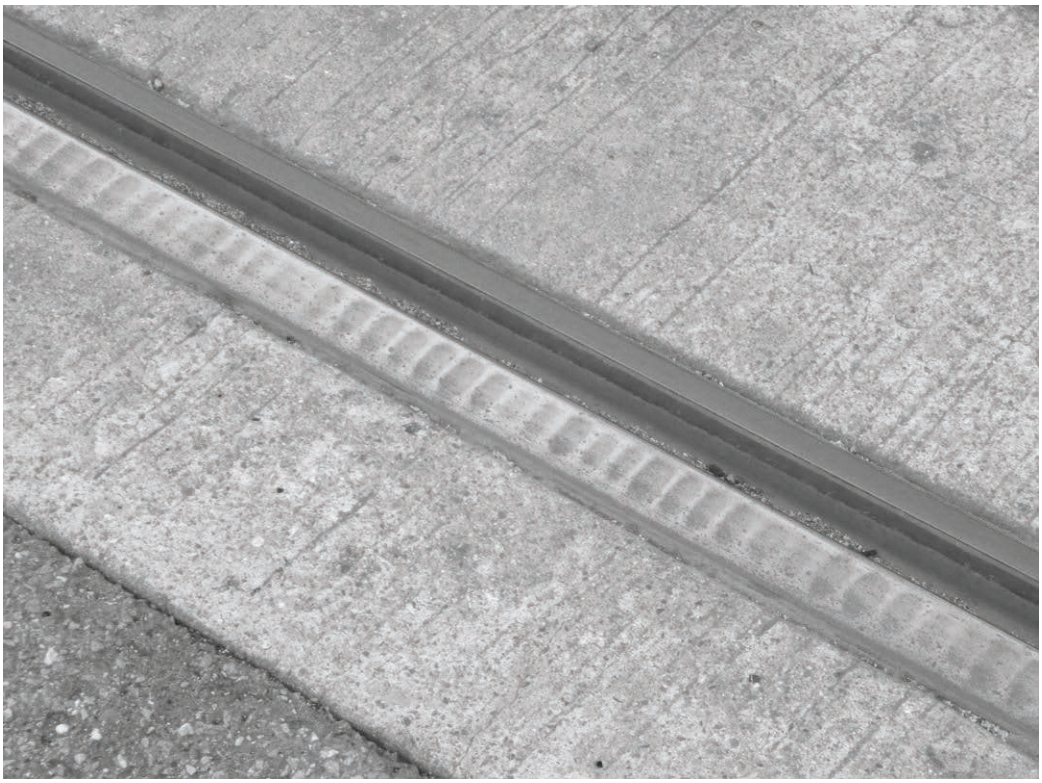

Figure 9.2.5 Corrugation on San Francisco cable car track

Very short pitch corrugation has been claimed at the Seattle Sound Transit system at both direct fixation track and embedded track. Similar observations have been observed at TriMet.

Rail corrugation is particularly problematic at the Detroit People Mover system, the TTC Scarborough Line, and the Vancouver Skytrain system, all of which employ linear induction motors (LIM) for propulsion, without tractive effort at the wheels. Most interestingly, anecdotal information suggests that rail corrugation may occur at areas where the LIM vehicle with non-powered wheels is accelerating![27] No satisfactory information has been obtained to explain this phenomenon. The return currents pass through a return rail, rather than through the track rails, so electrical heating does not appear to be a cause. The wheels of these vehicles are of small diameter, so that the contact stresses may be greater than would be the case with larger diameter wheels. Still, the vehicles are relatively lightweight. The self-steering truck is designed to promote curving and reduce squeal. Friction between the bolster and frame, however, may prevent full steering of the vehicle, even on tangents, thus promoting lateral slip. Spin slip was identified as the cause of corrugation at the Vancouver Skytrain system.

Corrugation mechanisms include spin slip of the tire about a vertical axis through the contact area, lateral oscillation of the tire, pinned-pinned resonances of the rail, primary suspension resonance, track resonance (or P2 resonance), torsional resonance of the wheel set, and periodic creep within the contact zone. Corrugation may occur on ballast-and-tie track, direct fixation track, and embedded track.

The literature is rich with various theories regarding rail corrugation. A summary of corrugation observations at heavy rail transit systems and one light rail system is provided in *TCRP Research Results Digest 26*.[28] **Table 9.2.1** provides a summary of corrugation types and formative mechanisms as described by Grassie.[29] Grassie indicates that all types of corrugation are essentially constant frequency phenomena, a hypothesis that is contrary to some earlier theories. The pinned-pinned mode of rail vibration is essentially a constant frequency phenomenon, related to discrete rail supports, a geometric characteristic. However, rail corrugation occurs at embedded track without discrete rail supports, indicating that other mechanisms besides pinned-pinned modes and discrete track supports may cause corrugation. Ciavarella and Barber have developed a theory of rail corrugation that is independent of pinned-pinned modes or other resonances, suggesting that rail corrugation wavelength may be induced by an existing periodic roughness in the running surface.[30] One prediction regarding rail corrugation can be made: corrugation is likely to occur if the rails are not ground regularly.

Table 9.2.1 Corrugation categories (After Grassie, 2010)[29]

Type	Wavelength fixing mechanism	Track type	Typical frequency – Hz	Damage mechanism
Pinned-pinned	Pinned-pinned resonance	Tangent and high rails	400-1200	Wear
Rutting	2nd torsional resonance of driven axles	Low rail	250-400	
Other P2 resonance	P2 resonance	Tangent or high rails	50-100	Wear
Heavy haul	P2 resonance	Tangent and curves	50-100	Plastic flow in troughs
Light rail	P2 resonance	Tangent and curves	50-100	Plastic bending
Trackform specific	Trackform specific	Tangent or curves	–	Wear

Rutting corrugation is related to torsional resonance of the wheel set. Rutting corrugation may not be relevant to light rail systems with resilient wheels. Not all light rail or heavy rail systems use resilient wheels, so rutting corrugation may yet be a significant factor at some systems.

The P2 resonance is the resonance of the wheel set and rail on the track support stiffness. This is also greatly modified by the resilient wheel, so one may speak of a resonance of the tire and rail on the track support, and the resonance of the axle set and wheel centers on the resilient wheel springs. The vibration is probably coupled such that the P2 resonance is not clearly defined for resilient wheels.

Track curvature influences corrugation rates. Dring reports that corrugation rates at wavelengths of 4 to 10 inches [100 to 240 mm] were measured at the MTRC Island Line in Hong Kong.[31] The results are listed in **Table 9.2.2**. These data suggest that at relatively modest curve radii of 1,300 feet [400 m] or less, corrugation depth might reach as much as 0.052 inch [1 mm] within as little

as 1 year. Corrugation rates were evidently reduced by 60 to 70% relative to 1992 values after full scale implementation of modified wheel profiles. These profiles were developed in combination with rail profiles to control both rail corrugation and shelling. The MTRC vehicle is a conventional rapid transit vehicle with solid wheels, and results for light rail vehicles with resilient wheels may differ. The MTRC rail corrugations are believed to be due to plastic deformation, related to high contact forces at track resonance frequencies governed by the wheel set unsprung mass and the track stiffness. Dring also reports short pitch corrugation at the MTRC, at wavelengths of 2 to 3.5 inches [50 to 90 mm]. The depth of these short pitch corrugations did not exceed 0.002 inch [0.05 mm]. These were prevalent on the low rail and characterized by "white" patches (inferred here to mean martensite, indicative of heating due to slip).

Table 9.2.2 Corrugation growth rates at MTRC Hong Kong for various track curvatures in 1992 (After Dring, 1994)[31]

Curve Radius		Corrugation Growth Rate	
M	feet	mm / week	0.001 inch/week
< 400	<1,300	0.025	1
400 to 1000	1,300 to 3,300	0.01	0.4
> 1000	> 3,300	0.001	0.04

New rails are evidently more prone to corrugation than older rails at the MTRC due to lack of work hardening,[31] which may require considerable periods of time. Even so, the MTRC axle loads are evidently high for rail transit. Rail grinding soon after rail installation would likely require additional grinding after work hardening has taken place. As discussed below, residual stress accumulation due to contact loading (shakedown) reduces the propensity for additional plastic deformation and thus might reduce the propensity for rail corrugation of the type described by Dring. Residual stress accumulation might be relatively rapid after installation.

Noise levels at the MTRC were observed to increase by 7 to 10 dB as the corrugation approached 0.016 to 0.020 inch [0.4 to 0.5 mm], providing an indication of the significance of corrugation amplitude. Thereafter, interior noise levels remained relatively constant with increasing corrugation depth (possibly due to loss of contact between the wheel and rail). At these corrugation amplitudes, the wheel and rail tend to separate from each other, if only for a millisecond at a time. (This may have some implication regarding signaling.)

9.2.1.1.4 Grinding Artifact Noise

Grinding artifact noise is caused by periodic grinding patterns left in the rail by rail grinding machines, and these patterns can be confused with corrugation. The noise produced by periodic grinding marks is similar to gear noise, such as the sound of worn differential ring-and-pinion gears in automobiles. While preferable to corrugation noise, such noise may be objectionable. The grinding artifacts are caused by irregular or excessively rough grinding wheels and high rates of metal removal due to coarse grinding at high grinding train traveling speed. Grinding procedures involving high grinding train traveling speed and single passes are most prone to leaving grinding artifacts. Grinding marks on head-hardened or alloy rail may not be easily worn

down by lightweight rail transit vehicles as opposed to freight trains. Rail grinding methods used on freight railroads may not be valid for rail transit.

Rail corrugation has been associated with grinding marks left by rail grinding machines at TriMet and Sound Transit.[32] Anecdotal evidence obtained at these systems suggests that residual grinding marks may trigger subsequent rail corrugation. As noted above, natural corrugation at very short wavelengths comparable with grinding mark wavelengths can occur. Some interaction may occur between periodic grinding artifacts and the tire, such that the grinding artifact amplitude may increase in severity, causing corrugation at the same wavelength. Ciavarella and Barber have developed just such a theory where existing periodic roughness may lead to rail corrugation at the same wavelength.[30]

9.2.1.1.5 Singing Rail

Singing rail is noise radiated by the rail at large distances ahead of and behind the train, and is due to rail bending waves propagating at velocities in excess of the velocity of sound in air. Singing rail is most apparent with soft direct fixation fasteners with low damping. Uniform spacing of rail supports also produces stop bands and pass bands in the bending wave vibration spectrum. Resilient rail pads or direct fixation (DF) fasteners that support the rail through elastomeric elements tend to decouple the rail from the rail support, broadening the spectrum of vibration transmission throughout the audio range. Singing rail is apparent at Bixby Knolls on the Los Angeles Blue Line in California, and at the Collingswood station near Philadelphia. The rail at Bixby Knolls is ballasted track with concrete ties, spring clips, and pads, at a 30-inch [750-mm] pitch. The rail at Collingswood station is supported by unconventional direct fixation with the rail head and web directly supported by elastomeric shoulders, leaving the base of the rail hanging free.

9.2.1.2 Factors Affecting Rolling Noise

Various factors and features of track design that influence rolling noise are described below. These factors may also influence ground vibration and ground-borne or structure-borne noise, although the frequency regime is much lower with structure-borne noise and vibration. The term “structure” refers to the trackbed or aerial structure, as opposed to the rail.

9.2.1.2.1 Wheel Dynamics

The dynamic response of the wheel set substantially affects high-frequency rail and wheel vibration and thus wayside noise. The response is affected by axle bending modes beginning at about 80 to 120 Hz, wheel and/or tire resonances above 400 Hz, and spring-mass resonances of resilient wheels.

Figure 9.2.6 illustrates the radial acceleration response of a typical resilient wheel. The first radial distortional mode of the tire is at about 500 Hz, followed by numerous radial modes at higher frequencies. One may expect peaks in wayside noise spectrum at these frequencies, related to noise radiation by the wheel.

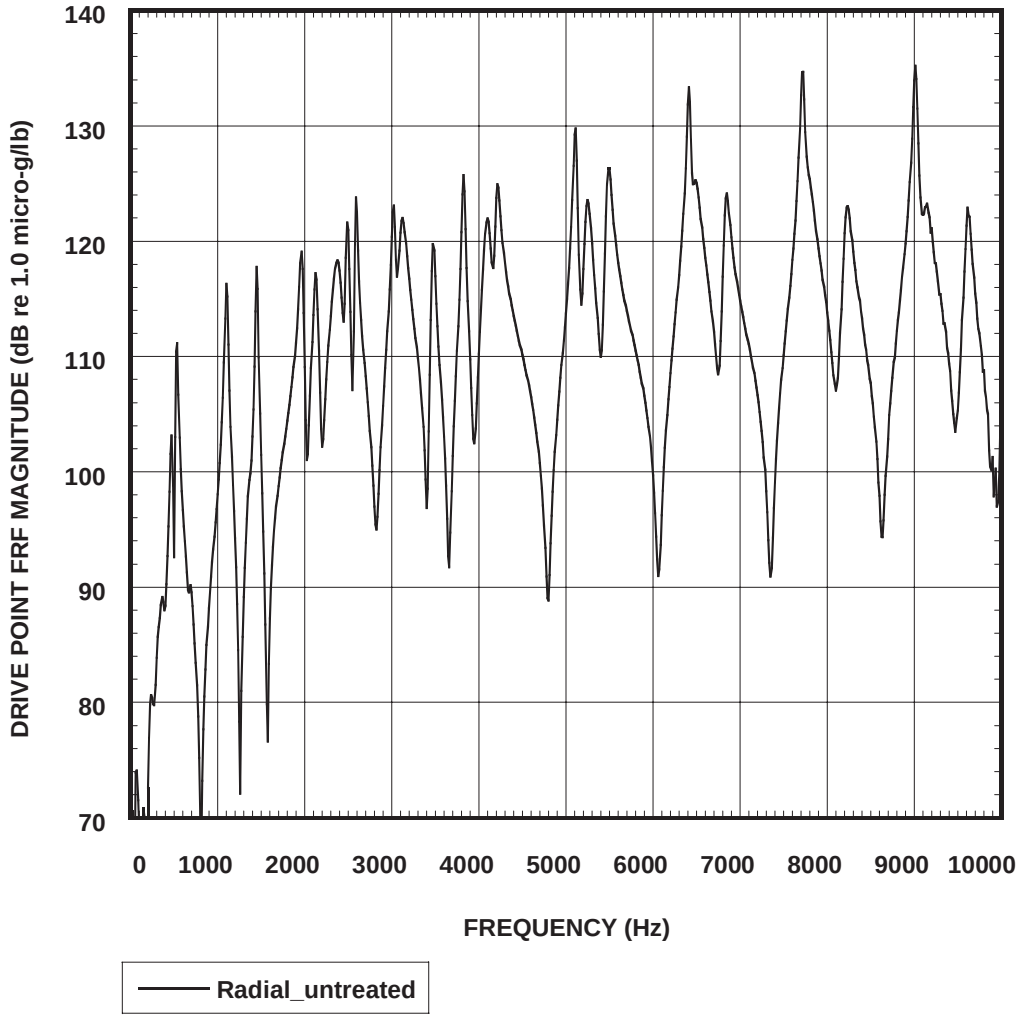

Figure 9.2.6 Radial mechanical acceleration response of TriMet Type 2 vehicle resilient wheel

Figure 9.2.7 shows the corresponding radial mechanical impedance of the tire of the same wheel. The mechanical impedance represents the reaction force of the tire to an input velocity and is directly related to the acceleration response shown in Figure 9.2.6. The minima of the radial mechanical impedance correspond to radial resonances of the wheel and tire. The maxima of the radial impedance correspond to anti-resonances at which frequencies maximum wheel reaction forces would occur. (The picture is actually more complicated, as the dynamics of the rail must be taken into account.) In this case, the highest peak in the mechanical impedance curve occurs at about 775 Hz, where the impedance is 5,000 lb-sec/in [0.875 KN-sec/mm]. Assuming a rail surface roughness of 0.001 inch [0.0254 mm], the corresponding reaction force of the wheel would be 24,350 lb [113 KN], exceeding the static load. The actual force would differ, due to the compliance of the rail, but one can see how a modest rail roughness or wheel flat can produce high contact forces, wear, and noise. Contact separation is a distinct possibility for track with rough running surfaces. Contact separation would tend to occur less at AW3 loading than AW0 loading, due to the higher static contact load.

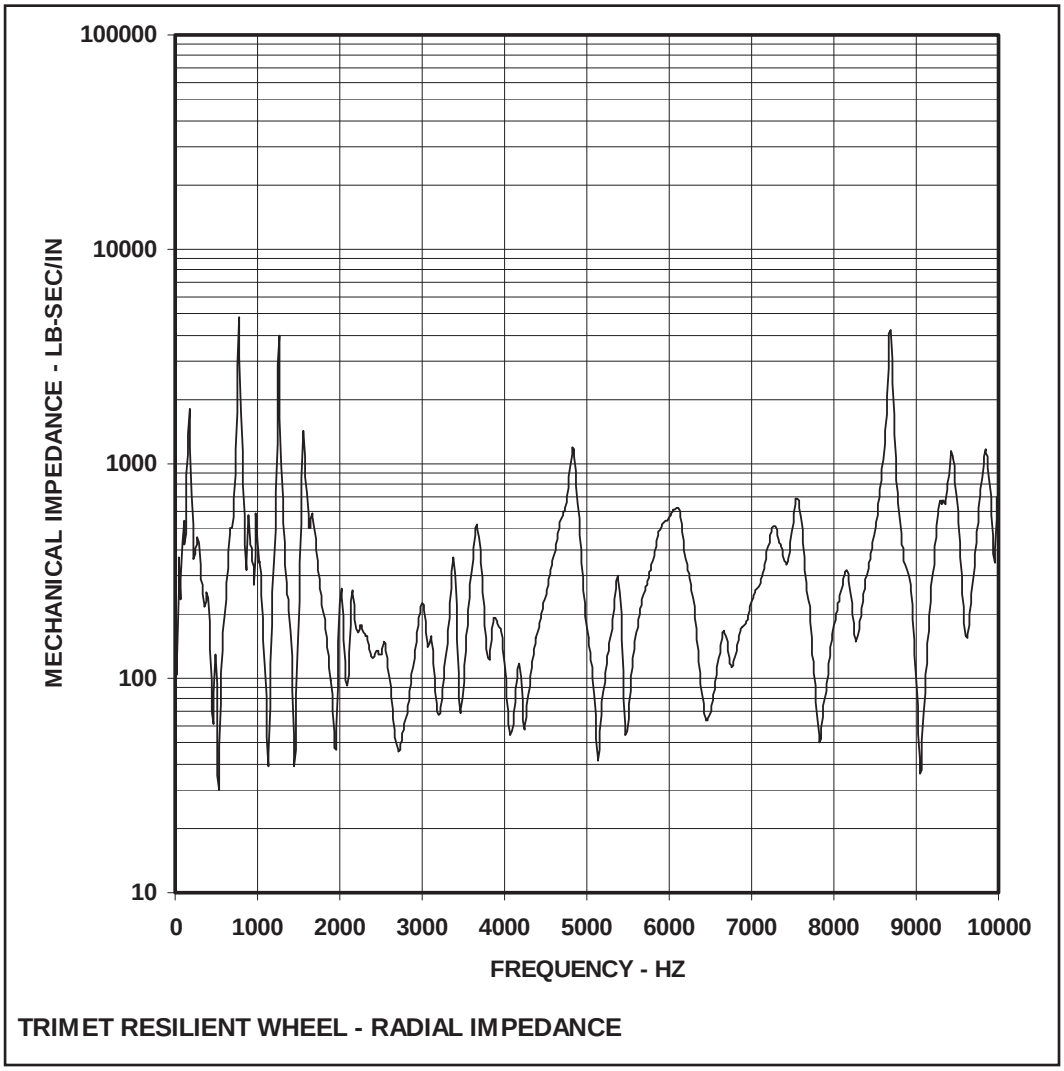

Figure 9.2.7 Input mechanical impedance of TriMet resilient wheel

9.2.1.2.2 Rail Dynamics

The dynamic response of the rail strongly influences wayside noise radiation. Up to about 400 Hz, the rail behaves as a simple beam on an elastic foundation. At higher frequencies, standing waves occur between the rail supports. The first of these is the pinned-pinned mode, where the wavelength of the standing wave is equal to twice the fastener pitch. The maximum amplitude of vibration occurs midway between the fasteners, and the minimum amplitude occurs at each fastener position. Estimates of the pinned-pinned mode resonance frequencies based on Timoshenko beam theory are presented in **Figure 9.2.8**. The pinned-pinned mode resonance frequencies of 115 RE rail supported at pitches of 36 inches and 30 inches [900 mm and 750 mm] are about 500 Hz and 750 Hz, respectively. The pinned-pinned mode resonance frequency increases with increasing rail size, but not rapidly.

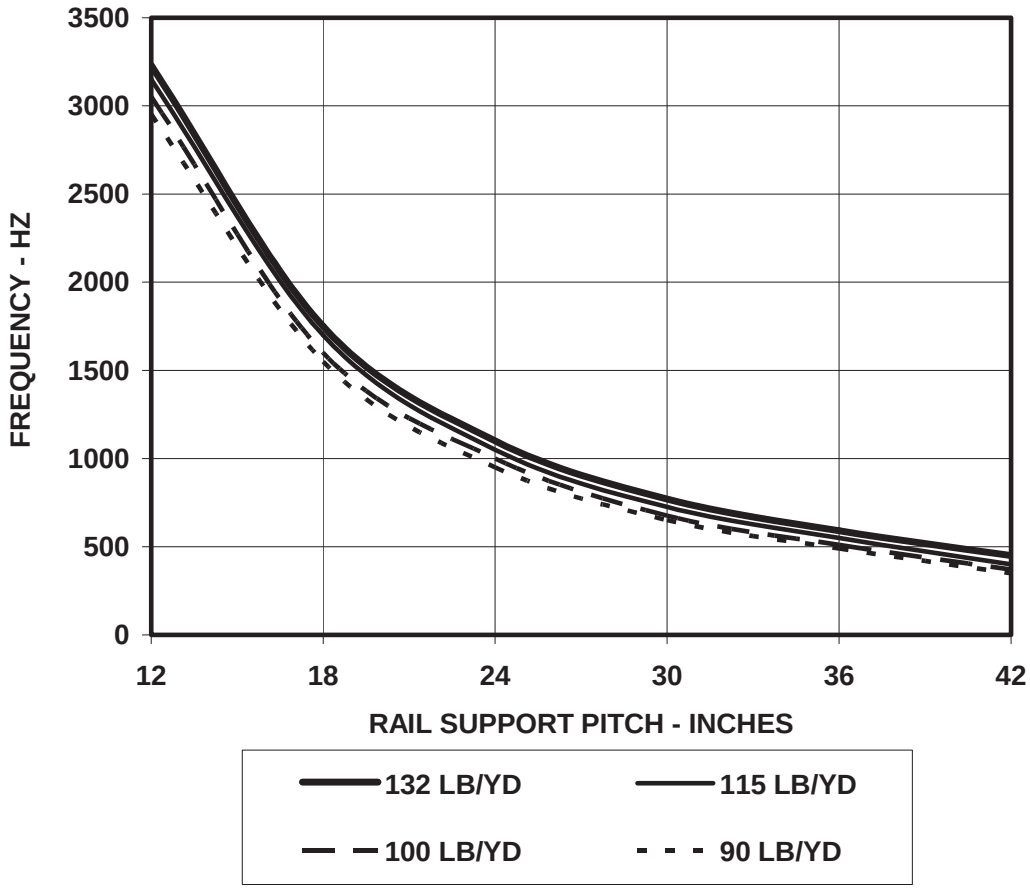

Figure 9.2.8 Vertical pinned-pinned resonance frequency vs. rail support separation for various rails

Grassie and Edwards have identified the pinned-pinned mode as being responsible for one form of short pitch corrugation.[33] However, short pitch corrugation has been observed at embedded track with continuous elastomer support, for which no pinned-pinned mode exists,[34] indicating that other factors may contribute to rail corrugation besides pinned-pinned modes. (These factors may include the vertical and lateral rigid body modes of the resilient wheel tire, tire running surface profile, torsional vibration of the axle and wheel centers, and perhaps other causes.)

Figure 9.2.9 illustrates the mechanical input impedance of the rail head for forces acting vertically against the head, based on exact solutions for a Timoshenko beam discretely supported at 30-inch [750-mm] spacing by fasteners with vertical dynamic stiffness of 250,000 lb/in [44 MN/m]. The first curve is the input impedance directly over the fastener, and the second is the input impedance at a point midway between fasteners. The measured mechanical impedance of the TriMet resilient wheel is shown for comparison. Both of the rail impedance functions contain a dip at about 100 Hz, which is the resonance of the rail on the fasteners, and would be predicted by a continuously supported rail with equivalent rail support modulus. The curves depart considerably above 200 Hz due to the discrete nature of the rail support. The rail's high-frequency input

mechanical impedance depends strongly on whether the input point is between the fasteners, over a fastener, or at some other point between these. Thus, as the wheel rolls over the rail, it sees a rapidly varying mechanical impedance at high frequencies depending on its position relative to the fasteners.

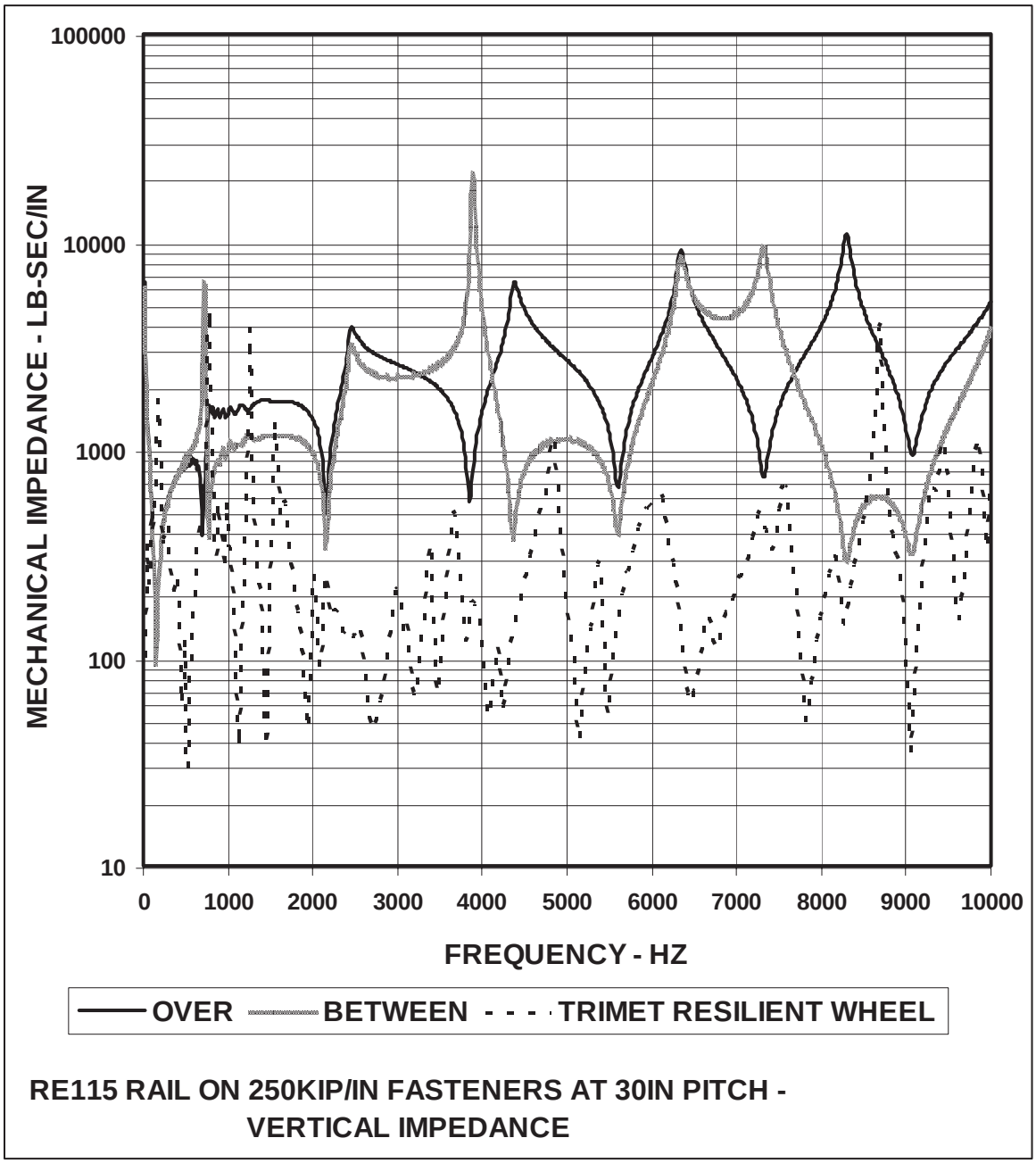

Figure 9.2.9 Mechanical input impedance of 115 RE rail on discrete supports

Except at a few isolated frequencies, the mechanical impedance of the rail substantially exceeds that of the wheel tire above roughly 2,000 Hz. The greatest opportunity for complex interaction of the wheel tire and rail occurs at frequencies below 2,000 Hz, especially near the pinned-pinned

mode. **Figure 9.2.10** is an expanded view of Figure 9.2.9, which shows how close the pinned-pinned mode is to a major anti-resonance of the resilient wheel.

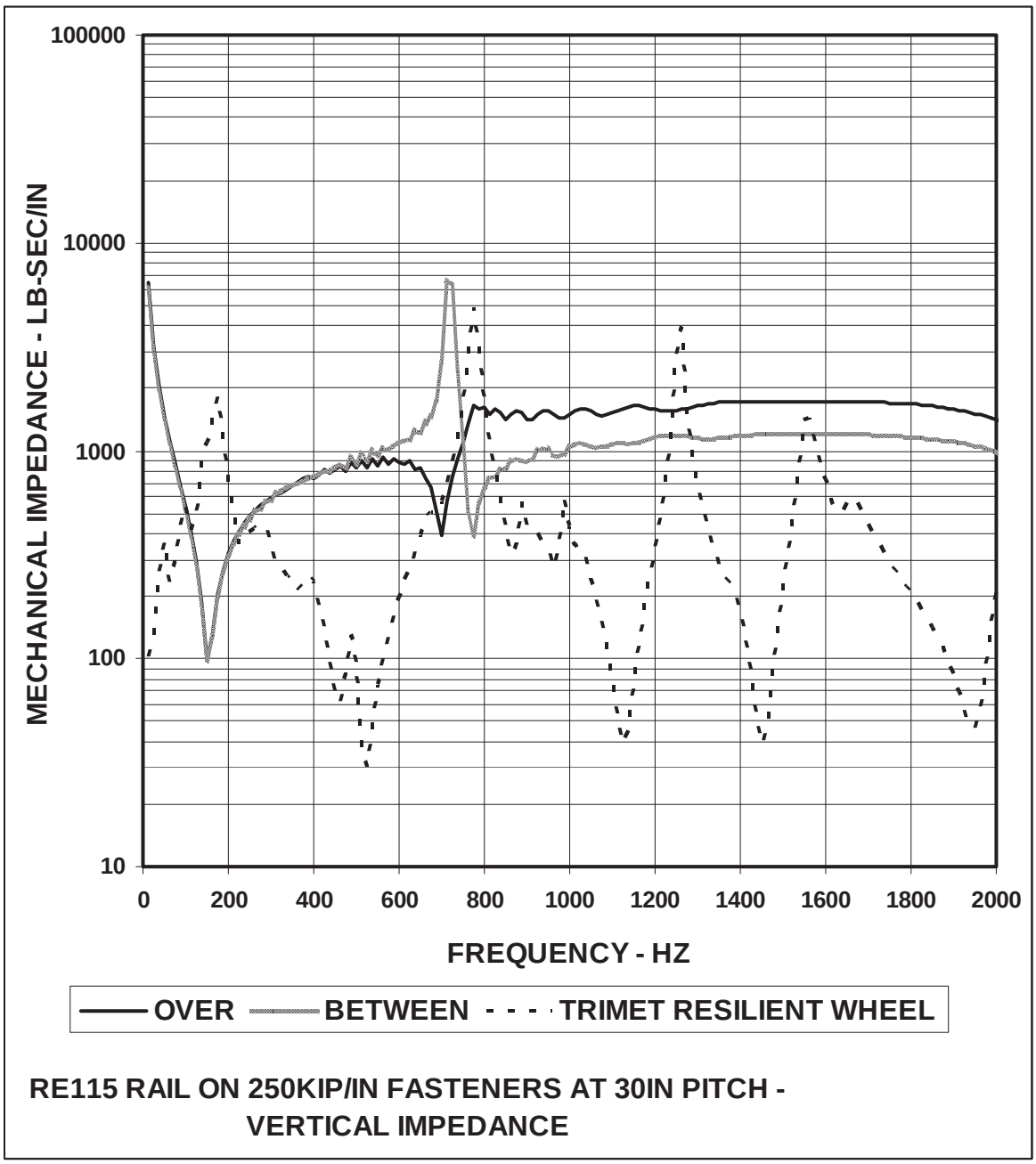

Figure 9.2.10 Input impedance of 115 RE rail (0 to 2000 Hz)

Timoshenko beam theory is excellent for analysis of high-frequency rail vibration up to about 1,000 or 1,500 Hz. Above 1,000 Hz, the vibration modes of the rail flange and web enter into the response. That is, the rail no longer bends as a simple beam, but "breaks up" into local modes, greatly complicating the response relative to those shown in Figures 9.2.9 and 9.2.10. The lateral vibration response of the rail is also very important, as distortional vibration of the rail web may occur at frequencies in the neighborhood of 1,000 Hz and above. The tire's vibration response is

similarly complicated. Thus, beam theory for detailed modeling of wheel/rail interaction at frequencies above 1,000 or 2,000 Hz may be of little quantitative value. Finite element analysis offers a much more complete picture of rail and wheel vibration at the cost of considerable computer time.

Figure 9.2.11 illustrates the theoretical vertical vibration response of the rail at a distance of about 50 ft [15 m] from vertical forces applied to the top of the rail—one force located at a point above the fastener and the other force located between the fasteners. This model corresponds to that represented in Figure 9.2.9 and Figure 9.2.10. The rail is 115 RE with 250,000 lb/in [44 KN/mm] fasteners at 30-inch (750-mm] spacing. Bending waves propagate in the rail at frequencies between the rail-on-fastener resonance frequency and about 700 Hz. Between this frequency and the pinned-pinned mode frequency of about 770 Hz, vibration transmitted along the rail is greatly attenuated, depending on the rail support dynamic characteristics and uniformity of the spacing, producing what is termed a "stop band." As shown in Figure 9.2.11, the stop band can be very dramatic; at 50 ft [15 m] from the source, the rail response is nil. Between the pinned-pinned mode frequency and the next higher cutoff frequency, bending waves may propagate freely in a "pass band." The theoretical transmission of vibration in the rail contains a series of stop bands and pass bands. The rail's ability to radiate noise will be affected by the widths of the stop and pass bands, and a slight randomness in the support separation may significantly alter the stop and pass band characteristics. Grassie indicates that introduction of randomness in the rail support is intuitively attractive, but suggests that pinned-pinned mode would move to those frequencies representative of the mean spacing.[29] However, the pinned-pinned mode is the result of periodic supports with uniform spacing, and introduction of randomness would tend to destroy the pinned-pinned mode and associated stop bands and pass bands of rail bending waves. Reducing the rail support pitch will increase the pinned-pinned mode frequency and those of the pass bands and stop bands, as noted above. (The ripples of the responses shown in Figure 9.2.11 are numerical artifacts related to the use of a finite number of rail supports, in this case, 256, and finite rail length.)

The main point here is that the response of the wheel and rail above 500 Hz is very complicated and that the propensity for adverse interaction between these components, leading to tonal components of wayside noise and possibly corrugation, is high. Track design should, ideally, be directed toward minimizing the chances of these interactions. Reducing the rail support spacing and introducing damping into the track support system should be useful for this purpose. Also, some types of direct fixation fasteners support the rail directly through an elastomer element, decoupling the rail from the top plate or base plate so that the rail is free to act as an infinite unsupported beam without pinned-pinned modes. A free infinite rail exhibits considerable damping due to rail vibration energy being radiated away from the source, and, as a result, might be less susceptible to rail corrugation. Axle spacing also affects the response of the rail. That is, bending waves reflect between the tires as the truck moves down the track. Selection of fastener spacing to avoid undesirable amplification of rail vibration between the wheel sets may be possible. To the extent that anti-resonances occur in the wheel tire, choosing the fastener pitch such that the first anti-resonance of the tire falls within the rail's stop band might be an attractive approach to controlling singing rail. Research in this area is needed to explore these design options.

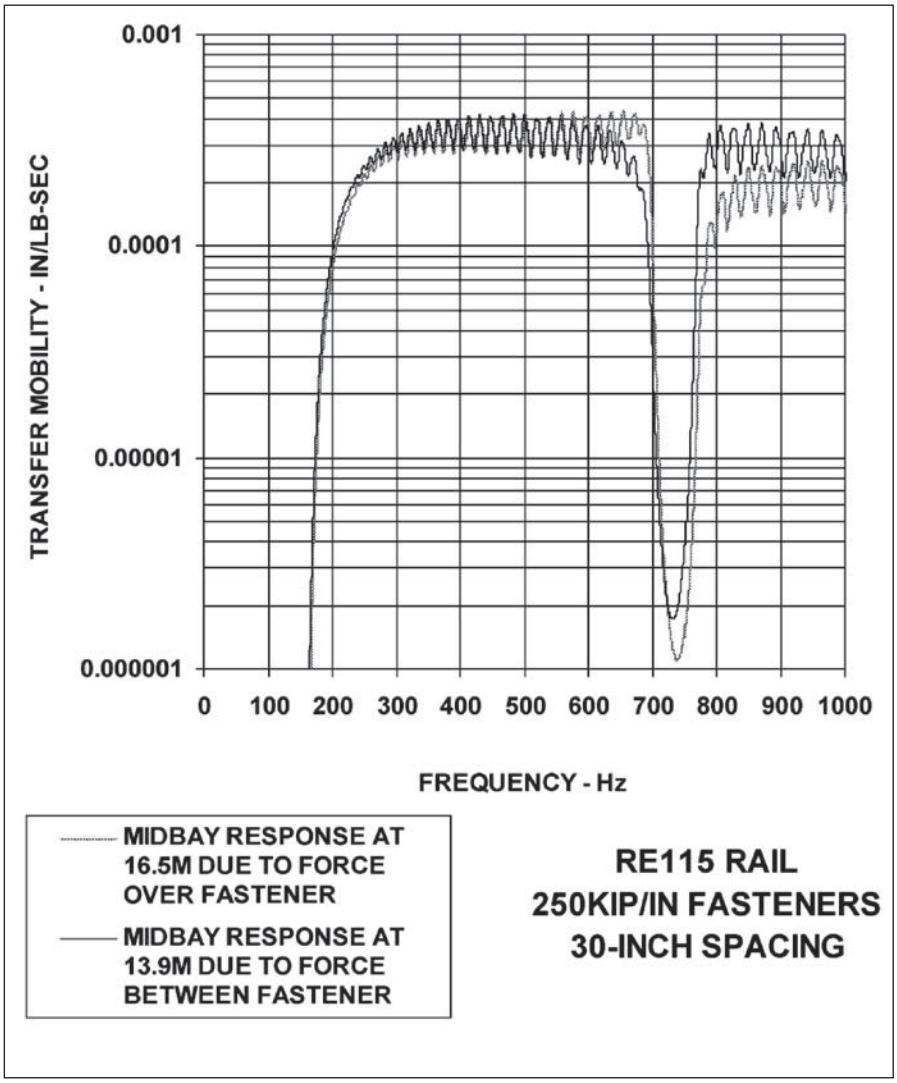

Figure 9.2.11 Theoretical vertical rail velocity responses at about 50 ft [15 m] from vertical forces directed against the rail head

9.2.1.2.3 Resilient Direct Fixation Fasteners

The input mechanical impedance of the fastener influences the response of the rail, as indicated in Figure 9.2.8, Figure 9.2.9, and Figure 9.2.10. In general, very stiff fasteners reduce the response of the rail at the expense of increasing structure-borne noise and vibration. Soft fasteners allow the rail to vibrate more than stiff fasteners do. Fasteners that support the rail with elastomer shoulders may have little effect on rail vibration at high frequencies (above perhaps 200 Hz), because the mechanical impedance of the elastomer decreases with increasing frequency. Soft fasteners with very massive top plates in direct contact with the rail tend to block the response of the rail at high frequencies, thus promoting the pinned-pinned mode. Resonances of the top plate may further affect rail response. To complicate matters more, the base of the rail, the clip, and the shoulder of the fastener combine to further influence the dynamic stiffness “seen” by the rail. Details concerning rail fastener performance characteristics are presented in Article 9.2.1.3.5.

9.2.1.2.4 Ballasted Track

Noise from ballasted track is usually less than from resilient direct fixation track, due to the acoustical absorption provided by the ballast. The character of wayside noise from ballasted track also differs significantly from that produced at direct fixation track, probably due to differing dynamic characteristics of the rail support and rail support separation, as well as the amount of trackbed sound absorption. For example, the cross ties at ballasted track may be placed with a slightly random separation that tends to destroy the pinned-pinned mode of rail vibration, where direct fixation track fasteners are usually installed at precise separations. Ballasted track with wood ties, tie plates, and cut spikes probably provides greater damping than track with concrete ties with rail clips or direct fixation fasteners. These factors tend to reduce reverberation (standing waves) of vibration in the rail, making the rail sound "dead" compared to direct fixation track.

9.2.1.2.5 Contact Stiffness

Contact stiffness is the ratio of the contact vertical force to the relative vertical deflection of the wheel tire and rail, assumed here to be the relative deflection of their respective neutral axes. The mechanical impedance of the contact stiffness is inversely proportional to the frequency. If the contact stiffness is small relative to the stiffness of the wheel or rail, wheel/rail forces will be controlled by the contact stiffness, such that a halving of contact stiffness would halve the dynamic forces driving the wheel and rail, thus reducing wheel/rail forces and noise by six decibels. The contact stiffness is typically about 6,000,000 lb/in [1,050 KN/mm] for a rail head radius of 10 inches [254 mm] and linear tread profile. The contact mechanical impedance at 1 KHz is

$$Z_c \text{ (1,000 Hz)} = 6{,}000{,}000 \text{ lb/in} / (2 \times 3.14159 \times 1{,}000 \text{ Hz})$$

$$= 955 \text{ lb-sec/in}$$

In S.I. units, this is

$$Z_c \text{ (1,000 Hz)} = 1.05\text{E}9 \text{ N/m} / (2 \times 3.14159 \times 1{,}000 \text{ Hz})$$

$$= 167{,}000 \text{ N/m or } 0.167 \text{ KN/mm}$$

This can be compared with the measured input mechanical impedances of the resilient wheel, shown in Figure 9.2.7, and that of the rail, shown in Figure 9.2.9. At 1,000 Hz, the impedance is comparable with those of the wheel and rail. Most importantly, the contact mechanical impedance is significantly less than that of the wheel at the anti-resonance frequencies of the tire. These anti-resonances produce peaks in the wheel/rail contact forces, and these forces are reduced if the contact mechanical impedance is less than the tire and rail mechanical impedance. The contact mechanical impedance decreases to 500 lb-sec/in [0.0875 KN/mm] at 2,000 Hz and 250 lb-sec/in [0.0438 KN/mm] at 4,000 Hz. Thus, the contact impedance becomes small relative to that of the rail at frequencies above 2,000 Hz, but remains comparable to the mechanical impedance of the wheel. The contact stiffness tends to dominate the rail response and reduce the vibration of the rail at these higher frequencies.

With high conformal contact, the contact stiffness may increase to 20,000 lb/in [3.5 KN/mm], with a contact mechanical impedance on the order of 3,000 lb-sec/in [0.525 KN-sec/mm], comparable with that of the rail. Thus, rail vibration at high frequencies should be high with high conformal contact! As it turns out, though, high-frequency wayside rolling noise above 4,000 Hz is usually less important than at 500 to 1,000 Hz, at least for a low degree of conformal contact. More research is needed, including field testing, to determine the effect of contact stiffness, or more specifically, conformal contact, on wayside noise.

The contact stiffness does not vary greatly over the range of rail head crown radii for a typical wheel with straight taper. The contact stiffness varies about 16% for head radii between 6 inches [150 mm] and 15 inches [375 mm]. Under the most optimistic scenario, reducing the rail head radius would decrease contact forces, and thus noise, by at most 1.5 dB, unless the starting point is a high degree of conformal contact. Thus, from the point of view of track design, the head radius should be sufficiently small to avoid development of fully conformal contact across the rail head for any reasonable wheel truing interval. The current AREMA standard for 115RE rail specifies a rail head crown radius of 8 inches [200 mm], which would be consistent with avoiding conformal contact at the running surface on tangent track. Conformal contact at low radius curves will occur at the gauge corner, as this is controlled by the wheel profile at the throat and the gauge corner radius, both subject to wear, and should not necessarily be avoided.

9.2.1.3 Treatments for Rolling Noise Control

Track-oriented treatments for controlling rolling noise are described below, with emphasis on those treatments that are common in the industry.

9.2.1.3.1 Continuous Welded Rail

Rolling noise levels with properly ground continuous welded rail and trued wheels in good condition are the lowest that can be achieved without resorting to extraordinary noise control measures. Impact noise from rail joints can be clearly audible with moderately maintained track. Noise from continuous welded rail may be as much as 5 dB lower than noise from jointed rail. Continuous welded rail requires less maintenance than jointed rail, so the benefits of low noise are more easily retained.

9.2.1.3.2 Hardened Rail

Hardened rail is less prone to rail corrugation and wear than rails of softer steel. Head-hardened and fully heat-treated rails are employed at curves and at station areas where acceleration/deceleration occurs. Hardened rail may prove beneficial for tangent track and at moderately curved track as well. Hardened rails should be carefully ground so as to avoid grinding artifacts that will not be worn down by subsequent vehicle operations. Further, excessive heating of the rail during grinding must be prevented to avoid loss of hardness. This is particularly important for work-hardened rail and/or rails with accumulated internal stresses due to wheel contact loading (shakedown). (See Article 9.4.4.)

9.2.1.3.3 Rail Grinding

Rail grinding combined with wheel truing is the most effective method for controlling wheel/rail rolling noise. With ground rail and trued wheels, wheel/rail noise levels at tangent ballasted track are comparable with the combined noise levels from traction motors, gears, and fans.[35] Rail grinding and rail grinders are discussed in detail in *TCRP Report 23*.[36]

Initial Grind

Even though rail grinding is usually the task of the transit system operator, an initial grind may be performed after track construction to remove mill scale from the rail for better traction and electrical conductivity. The grinding must establish a smooth finish and must maintain the head profile. After run-in and shakedown stress accumulation, the rail may be ground a second time to reestablish a smooth running surface. The second grind need not be aggressive or require significant metal removal, as it would essentially be a dressing operation.

If the rail is not ground initially, high contact forces due to roughness may occur, producing irregular hardness that may be difficult to smooth out over time. Initial grinding to remove scale and smooth the rail will provide a quiet system from start, also minimizing adverse community reaction to noise.

Overheating

Aggressive grinding may reduce running surface hardness by overheating. Thus, metal removal should not be conducted at such a high rate that it heats the rail excessively. Multi-stone grinders may avoid overheating. Any grinding program should be reviewed carefully with grinding contractors and engineers to ensure that head hardness is preserved.

Grinding Marks

Rail grinding should be done in a manner that does not introduce deep grinding marks into the rail. This is particularly important for hardened rail, as subsequent vehicle operations may not wear away the grinding marks. Some claim that grinding marks are acceptable, as they will be worn away over time. This is not likely to occur with head-hardened rail and light transit loadings. Grinding artifacts can be minimized with multiple grinding facets that closely mimic the desired crown and gauge corner radii, fine grit stone, and low translation grinding train speed. Grinding should ideally provide a smooth finish.

Grinding with an eight-stone grinder evidently reduced the amplitude of grinding marks at TriMet. Sixteen-stone grinders should provide a smoother finish than would four- or eight-stone grinders, given the same number of passes.[37] An eight-stone or sixteen-stone grinder can accomplish the same profile and finish with fewer passes than a four-stone grinder can. Grinding involving only a single pass with a four-stone grinder should be avoided.

Grinding procedures that result in a wavelength that produces a characteristic frequency during train passage that is comparable with the pinned-pinned mode frequency of the rail should be avoided. For typical light rail systems with a maximum speed of 55 mph, the translational speed is 968 in/sec [24.6 m/sec]. The pinned–pinned modal frequency of the rail at 30-inches [750-mm] pitch would be about 800 Hz, corresponding to a wavelength in the rail of about 1.2 inches [30 mm]. Grinding artifact pitches should be substantially less than this, and a conservative criterion would be 0.6 inch [15 mm]. However, short pitch corrugations with this wavelength have been noted at systems such as Sound Transit, suggesting that further investigation of the relationship between grinding procedures and corrugation rates should be conducted.

Rail grinding should be conducted in a manner that avoids coincidence between the rail grinding pattern pitch and rail corrugation wavelengths. Grinding train speeds, grinding wheel rotation speeds, and chronic corrugation frequencies should be reviewed before grinding. Rail grinding with a finish grind, or "acoustic" grind, with narrow facets and very short grinding pitch avoids coincidence with corrugation wavelengths.

A persistent howling noise developed at TriMet's ballasted track after grinding with the four-stone grinder. The cause may have been related to speed and grinding depth. An eight-stone grinder was brought in and the rail was reground to profile with 2 to 3 passes at 2.5 mph [4.0 km/h] pass rate with fine grit stones. The howling noise was eliminated. TriMet's position now is to specify an eight-stone grinder for all future grinding.

Grinding at Curves

Rail grinding at curves is desirable to control surface fatigue and defects, especially where lubrication is used. Lubricants can be forced into surface fatigue cracks under high stress and further open these fatigue cracks. This problem is probably greatest for heavy haul freights, but may occur at transit systems with soft rail.

System Layout

Rail grinding strategies should be considered when laying out track. Staging locations, such as pocket tracks and turnouts, for storing grinding equipment (as well as other track maintenance equipment) should be provided to minimize travel time. Grinding can be performed only if there is adequate access to the track during non-revenue hours or by single tracking, and grinding time can be maximized by minimizing travel time to and from the grinder storage location and the treatment section. Some grinders may have difficulty negotiating curves in tunnels or may be unable to grind rail on very short-radius curves. Adequate clearance must be included in track and structure designs to accommodate rail grinding machines. The ability to grind rail is a major factor in track layout and design.

Rail Profile

The optimal grinding procedure includes grinding the rail to achieve a head profile crown radius of about 8 to 11 inches [200 to 450 mm]. (The AREMA specification for 115RE is 8 inches [200 mm].) This should ideally be achieved with grinding facets of about $^{1}/_{16}$ inch [2 mm]. The rail head curvature has to be small enough that conformal contact of the wheel and rail would not occur for the maximally worn wheel. A rail head crown radius of as little as 8 inches [200 mm] may be appropriate for systems with infrequent wheel truing to avoid conformal contact.

TriMet's grinding approach for ballasted track does not include grinding to a precise ball radius, but grinding facets on the top of the rail at the gauge side and field sides, and then grinding the crown. This leaves grinding marks, but wear over a reasonable length of time eliminates the grinding marks and produces an approximate ball radius. TriMet has conducted very precise profile grinding with good surface finish, but has found this to be time consuming and impractical. Grinding time is limited to between 1 and 4 a.m., which does not leave much "spark" time. (Grinding marks may not wear away with time on tangent track with head-hardened rail.)

TriMet grinds the rail flat at embedded track sections, due to the difficulty of grinding a profile without the stone contacting the pavement or tram shoulder. This does not cause a problem with noise or steering, as train speeds are low on embedded track (on the order of 15 mph in downtown areas).[38]

Grinding Machines

Sixteen-stone grinders reduce the grinding time necessary to produce the desired contour relative to the time required with four-stone or eight-stone grinders. Computer-controlled grinders with various grinding profiles stored in memory simplify setup and further reduce grinding time. Grinding car speeds should be low enough to reduce the wavelength of grinding patterns to perhaps 0.25 inch [6 mm]. However, the speed should not be so slow as to excessively heat the rail and reduce its hardness. Grinding pressure and grinding train speed must be considered in developing a grinding program.

Vertical axis grinders with offset axes may be needed to grind embedded grooved (girder) rail. Horizontal axis grinders are capable of grinding grooved rail, but are not necessarily suitable for establishing good rail profile. Using standard tee-rail sections at embedded track might provide the greatest flexibility with respect to grinding embedded curves, although even here the presence of pavement may interfere with grinding.

Grinding Intervals

Periodic track inspections for corrugation growth and noise increase should be conducted to identify appropriate grinding intervals. A grinding interval equal to the exponential corrugation growth time (time for corrugation to grow by 167%) gives a rough estimate of the optimum grinding interval that minimizes metal removal. Portland's TriMet, for example, tries to grind rail every 3 years.[39]

TriMet attempts to grind all rails every 2 to 3 years or as needed to remove corrugations.[40] Normally, two to four passes are made with a four-stone grinder to recover a rough rail profile and establish a 0.75-inch [19-mm] wide contact strip; the last pass is made at a low speed to reduce grinding marks.

Rutting

Rail head profiles and tread profiles must be designed to work together. Kalousek and Johnson suggest that varying the location of the contact zone on the rail head may reduce rutting of the wheel tread and thus reduce wear that might greatly increase conformal contact and spin slip on tangent tracks.[41]

Conformal Contact

At curved track, the preference is for single-point contact of the wheel tread and rail, which necessarily requires that the rail gauge corner radius must be less than the radius of the tire throat. The wheel and rail will tend to wear together over time, producing conformal contact at the gauge corner and tread throat. That is, if the wheels are trued regularly, so that the rail sees the same wheel profile for all vehicles, the gauge corner should wear to the wheel's throat profile. This may reduce contact stresses and reduce gauge face wear and fatigue. Single-point contact

will promote a rolling radius differential with tapered wheel profiles, thus improving curving performance and reducing wear.

High wheel/rail conformity on tangents, caused by a large rail head radius and/or hollowed wheel treads, may promote spin slip corrugation.[41] Also, as discussed in Article 9.4.3, high conformal contact greatly increases contact stiffness, increasing contact forces between the tread and rail, and thus wear and noise. High conformal contact should be avoided on tangent track regardless of wheel wear and tread hollowing, requiring an effective wheel truing program. If wheel tread profiles were always linear, the rail head profile could be 11 to 14 inches [280 mm to 356 mm], giving a well-rounded contact area and low contact stiffness. However, tread wear, not necessarily producing false flanging, will increase conformal contact, as discussed in Article 0, so that a smaller rail head crown radius is needed in the absence of an aggressive wheel truing program. The crown radius should not be less than 8 inches [200 mm], otherwise rutting of the wheel will be exacerbated. See Article 9.4.1 for a discussion of contact dimensions versus rail head radii and wheel tread negative profile radii.

Rail Roughness Monitoring

Measurement systems are available for quantifying rail corrugation amplitude and wavelength, rail roughness, and grinding finish. These instruments should be capable of measuring corrugation amplitudes with a resolution of 4 microinches [0.1 micro-m]. The shortest corrugation pitches are on the order of 0.5 inch [12.5 mm]. Thus, the instrument should be capable of documenting wavelengths as short as 0.2 inch [5 mm]. Corrugations may have relatively long wavelengths. However, those that contribute to wayside noise and ground-borne noise would likely have wavelengths on the order of 4 inches [100 mm] or less. An instrument should be capable of measuring wavelengths of at least this length, and preferably to 40 inches [1,000 mm]. Measuring wave undulation at longer wavelengths is generally difficult and may require a laser profilometer.

9.2.1.3.4 Rail Support Spacing

Grassie suggests that reduction of fastener pitch may help control rail corrugation related to the pinned-pinned mode of the rail.[42] The pinned-pinned mode frequency is controlled by fastener spacing, or pitch, and is typically 800 and 500 Hz for fastener pitches of 30 and 36 inches [750 to 900 mm], respectively. Reducing the fastener spacing to 24 inches [600 mm] would drive the pinned-pinned mode resonance frequency well above 1,000 Hz, where the associated wavelength would be short enough to allow the contact patch to smooth out incipient short-pitch corrugation and thus reduce corrugation rates. (This has yet to be demonstrated.) The contact impedance decreases inversely with increasing frequency, so that increasing the pinned-pinned mode frequency may help the contact “spring” to isolate the tire and rail at this frequency. (Again, this has yet to be demonstrated.) In view of the above, direct fixation fastener pitch of 24 inches [60 cm] would be ideal, but has to be balanced against cost. However, the cost of the fastener could be lessened because the load requirements would be less than those of fasteners designed to support the rail at larger pitches. Installation costs, however, would go up.

Irregular spacing of fasteners, perhaps at 30 +/- 3 inches [750 +/- 75 mm], will reduce the pinned-pinned resonance effect. (This may explain why rail corrugation is less apparent at ballast and tie track relative to direct fixation track.) The degree of randomization need not be great, perhaps as

little as 5 or 10%. Langley discusses the similarity between material damping and a slight randomization of spacing between periodic supports of a beam on its forced response.[43] Allowing a slight variation of fastener spacing may allow the designer to be more flexible in laying out reinforcing bar, thus simplifying track design. Not replacing fasteners at precisely the same location as original fasteners would also simplify retrofit while accomplishing some randomization.

Track designers tend to locate fasteners on either rail directly opposite of each other. The result of this is that the wheels of the wheel set pass over the fasteners or between fasteners simultaneously. The effects of this on wheel/rail noise are not known, although parametric forces at both rails due to variation in rail head stiffness would be in phase and thus efficient producers of ground vibration at the fastener passage frequency. Locating fasteners in an alternating pattern would avoid in-phase parametric-related force transmission to the invert. No noise control benefit is apparent for maintaining fasteners exactly opposite each other. Thus, in curves, fasteners can be installed at nominal spacing without worrying about keeping fasteners in line.

9.2.1.3.5 Direct Fixation Fastener Design

Resilient direct fixation fasteners support the rail and provide modest vibration isolation, depending on stiffness and dynamic characteristics. The most common form of resilient direct fixation fastener consists of top and bottom steel plates bonded to an elastomer pad. Early designs feature rolled steel top and bottom plates, possibly with the anchor bolt extending through the top plate for lateral restraint. Modern designs incorporate forged top plates with anchor bolts that anchor the bottom plate, so that the top plate is retained by the vulcanized bond between the elastomer and metal components, and by the interlocking geometry of these components. The top plate provides shoulders that both confine the rail laterally and retain the rail clips.

Looseness

Airborne rattling noises due to looseness between the rail and its support are also eliminated with the typical bonded direct fixation fastener employing spring clips. For example, resilient elastomeric direct fixation fasteners reduced wayside noise from NYCTA steel elevated structures relative to levels for conventional timber tie and cut-spike track, much of which was due to eliminating looseness in the rail support.

Stiffness

Soft fasteners allow greater rail vibration and transmission of vibration along the rail than stiff fasteners, promoting “singing rail.” Soft natural rubber fasteners with low loss factor (damping) promote efficient propagation of bending waves that radiate noise. Wayside airborne noise levels may be increased by about 6 dB per halving of fastener stiffness, notwithstanding other factors. However, soft fasteners are useful for controlling structure-borne noise and vibration. An elastomer with a high loss factor will absorb rail vibration energy, thus reducing noise radiated by the rail. Neoprene is an attractive elastomer for this purpose, and neoprene has the added advantage of resistance to ozone and oils. Neoprene may be the preferred elastomer for direct fixation fasteners destined for concrete aerial structures or slab track at grade. However, neoprene should not be used where vibration isolation is required to control structure-radiated or ground-borne noise. Where vibration isolation is needed more than airborne noise control, such as on steel elevated structures or in subway tunnels, natural rubber is the preferred elastomer,

providing a dynamic-to-static stiffness ratio of less than 1.4. Thus, the selection of rail fastener stiffness and damping should be based on whether the track will be on an aerial structure, at grade, or in a tunnel.

Top Plate Stiffness

The effects of fastener top plate bending on rail-radiated wayside noise has not been experimentally determined. However, significant absorption of rail vibration might be expected at the top plate resonance frequency. Introduction of damping into the system and exploiting the top plate resonance may be beneficial in reducing pinned-pinned resonances. However, quite the opposite is suggested by Grassie, who provides an example of short-pitch rail corrugation related to top plate resonance.[42] Also, see the discussion regarding top plate design in Article 9.2.1.3.5, for an illustration of top plate bending vibration at high frequencies. Also, see the discussion below regarding ¼-wave resonance absorption.

Rail Pads

RailCorp in Sydney, Australia, uses rail pads with direct fixation track (which includes high-compliance fasteners). No corrugation was observed on direct fixation or ballasted track on the RailCorp system in Sydney, although RailCorp has indicated that they are concerned with corrugation.[44] Rail pads are used extensively in Europe and East Asia for both tie and ballast track and resilient direct fixation track. The rail pad may damp the pinned-pinned mode vibration in the rail, as well as decouple the rail from the fastener's top plate mass at high frequencies. Jones and Thompson, referring to a paper by Hempelmann, suggest that reducing the rail pad stiffness can reduce the probability of short pitch corrugation, especially directly over the tie.[45] [46] Grassie suggests that rail corrugation is affected very little by resilient rail pads unless the rail pad provides the bulk of the resilience of the rail support.[29] Rail corrugation related to the selection of rail pad stiffness occurred on resiliently supported bi-block tie track in Baltimore. These examples are not directly related to direct fixation, but they suggest that the rail pad might influence the pinned-pinned mode, especially for very stiff direct fixation fasteners or for fasteners with a massive top plate. This can be investigated easily by laboratory testing as well as numerical modeling. The standard pitch for direct fixation track in Sydney is 24 inches [600 mm], considerably shorter than the 30-inch [750-mm] pitch used in the United States. The pinned-pinned mode frequency is significantly higher with 24-inch [600-mm] spacing than with 30-inch [750-mm] spacing. One can see how a combination of short rail support pitch and damping may have favorable performance characteristics.

Quarter-Wave Resonance

An elastomer will exhibit a resonance due to standing waves propagating in the elastomer at some velocity determined by Young's modulus of elasticity and shape factor. In the case of elastomer in shear, such as with the so-called Cologne Egg fastener, the propagation velocity would be controlled by the shear modulus of the elastomer. A downward motion of the elastomer will induce a wave in the elastomer, and this wave will be reflected back from the base of the elastomer. The reflected wave arriving at the top of the elastomer will be in phase with the downward motion that produced the wave at the "quarter-wave" resonance frequency, producing a resonance dip in the mechanical impedance as seen by the top plate and/or rail. At this frequency, absorption of rail vibration energy reaches a maximum. This feature has been

suggested by Remington as a rail noise control if the thickness of the elastomer is large enough to reduce the quarter-wave resonance frequency down to perhaps 500 to 1,000 Hz.[47] The thickness of the elastomer would have to be on the order of 1 to 2 inches [25 mm to 50 mm], however, for a compression mode fastener. Elastomer in shear would allow a smaller elastomer thickness.

9.2.1.3.6 Trackbed Acoustical Absorption

Ballasted track is well known to produce about 5 dB less wayside noise than direct fixation track due to the sound absorption provided by the ballast and differences in the track support characteristics. Acoustically absorptive concrete or glass-fiber panels placed very close to the rail may reduce noise by perhaps 3 decibels when installed on direct fixation track. Such panels would have no effect on ballasted track because the ballast already provides substantial acoustical absorption. Track inspection and maintenance must be considered before applying this treatment.

9.2.1.3.7 Tuned Rail Vibration Absorbers

Rail vibration absorbers are tuned resonant mechanical elements that are attached to the rail base and web to absorb vibration energy and thus reduce noise radiation by the rail. Substantial clearance beneath the rail may be required for installation. The absorbers can have multiple elements, providing a multi-degree-of-freedom system with highly damped modes tuned to a number of frequencies. Vibration absorbers may be impractical on ballasted track unless they can be positioned clear of the ballast to maintain electrical isolation. If electrical isolation is not a factor, piling the ballast above the rail base should provide substantial absorption without vibration absorbers. The absorber is effective where the track exhibits little damping, such as at ballasted track with concrete cross ties, resilient rail pads, and clips or at direct fixation track.

Data provided by some manufacturers indicate a reduction of about 3 to 5 dB in rail vibration at ⅓-octave band frequencies between 300 and 2,000 Hz for 70-mph [111-km/h] trains on tangent track with absorbers mounted on each rail, one between each rail fastener. The mass of each absorber was 50 lb [23 Kg].

Tests conducted at Portland's TriMet under TCRP Project C-03A indicate that wayside and undercar noise was actually about 1 to 2 dB higher with rail vibration absorbers than without. These tests are definitive, as they were conducted before and after installation of the absorbers. In spite of the adverse result, qualitative observations by TriMet engineers and others were that the noise during passage of light rail vehicles was less with the absorbers than without. That is, the absorbers produced an apparent noise reduction by eliminating the propagation of waves in the rails within the pass band frequency range defined by the discrete rail supports. That is, the sound was less reverberant. This particular track consisted of 115 RE rail with concrete ties, rail pads, and clips.

The quantitative noise reduction attributable to rail vibration absorbers may be nil where the wheel and vehicle auxiliary equipment dominate the wayside noise. This would most likely arise on ballasted track, which provides substantial acoustical absorption.

One may expect a qualitative reduction of rail reverberation and "singing rail," both related to pinned-pinned modes. As such, they may be particularly attractive in reducing the perception of

noise and improving the acceptance of transit by the public and may also have benefits in controlling rail corrugation, which has obvious long-term maintenance cost benefits. Thus, rail vibration absorbers might be considered for treatment of chronic rail corrugation at curves or other problematic sections of track.

The main disadvantage of rail vibration absorbers is increased maintenance costs related to the clips, nuts, and bolts required to clamp the absorbers to the rail. Water retention and possible corrosion should be of concern. Inspection of the rail may be impeded. The cost of rail vibration absorbers may be comparable with that of resilient direct fixation fasteners.

9.2.1.3.8 Rail Vibration Dampers

Rail damping treatments are elastomer pads or sheets that bear against the rail flange, web, or both. They differ from rail vibration absorbers in that they do not have a vibrating mass; hence, they are not "tuned" to specific frequencies. They are most effective, however, at controlling high-frequency distortional mode resonances of the rail, including specifically the base and web. They may also be effective at controlling bending resonances of the rail, specifically the pinned-pinned mode. As with tuned vibration absorbers, the quantitative noise reduction attributable to rail vibration dampers may be nil, especially if the wheel-radiated noise and vehicle auxiliary equipment noise dominate the wayside noise. However, one may expect a qualitative noise reduction of rail reverberation and "singing rail." As such, rail vibration dampers may be particularly attractive in reducing the perception of noise and improving the acceptance of transit by the public. They may also have benefits in controlling rail corrugation, which has obvious long-term cost benefits.

The main disadvantage of rail vibration dampers includes obscuring the rail for the purpose of inspection. In areas where the track is not exposed to continuous moisture or drains freely enough that corrosion may not be significant, the rail vibration damper may be attractive. The damper should drain freely and not collect water against the rail and promote corrosion.

9.2.1.3.9 Wear-Resistant Hardfacing

"Hardfacing" is the application of a metal alloy inlay to the rail head by welding. The procedure involves cutting or grinding a longitudinal groove in the rail surface and welding a bead of the alloy into the groove. The Riflex welding technique has been used on a limited basis in the United States, primarily for wear reduction, but has been promoted in Europe since the early 1980s for rail corrugation control and wheel squeal. For additional information on Riflex welding, refer to Chapter 5, Article 5.2.5 in this Handbook.

9.2.1.3.10 Low-Height Sound Barriers

Low-height barriers placed very close to the rail have been explored in Europe for controlling wheel/rail noise, perhaps just outside the wheel's clearance envelope. In one extreme case, an aerial structure has been designed to provide a trough in which the vehicle runs, blocking sound transmission to the wayside.[48] Acoustical absorption is used to absorb sound energy before it escapes to the wayside. The height of the barriers must be determined by careful analysis. A 1- to 2-inch [2.5 to 5 cm] thick glass-fiber or mineral wool blanket or board with perforated protective cover should be incorporated on the rail side of the barrier. Adding sound absorption to the concrete slab surface of direct fixation track can be effective, but maintenance and access for

inspection should be considered. Low-height barriers placed immediately adjacent to the rails may interfere with rail grinding. Thus, they should be demountable to allow rail maintenance.

9.2.2 Special Trackwork Noise

Special trackwork includes switches, turnouts, and crossovers. The noise generated at special trackwork by wheels traversing frog gaps and related connections is a special case of impact noise. Impact noise is generated at turnouts as the wheel passes the switch point, the joint between the closer rail and the frog casting, the frog gap, and the joint between the frog casting and following rail. Impact noise can be minimized by eliminating joints and by maintaining the frog.

Various frog designs are used in transit track: solid manganese, flange bearing, lift over, rail-bound manganese, spring, and movable point frogs. Special frogs, including movable point, swing nose, and spring frogs, have been developed to minimize impact forces by eliminating the fixed gap associated with the frog, and special frogs can be a practical noise control provision for many transit systems. For additional information on frog design, refer to Article 6.6. The following guidelines are provided for frog design selection for noise control.

9.2.2.1 Solid Manganese Frogs

Solid manganese frogs with welded toe and heel joints provide an almost continuous running surface except for the open flangeway. Coordinated wheel and frog design along with effective track maintenance and wheel truing should provide adequate low-noise operation. Hollow, worn wheels with false flanges will contribute to noise and vibration when traversing through the frog.

9.2.2.2 Flange-Bearing Frog

Flange-bearing frogs with welded toe and heel joints are similar to the solid manganese frog design except that the frog supports the wheel flange while traversing the flangeway opening in the frog point area. The depth of the flangeway is reduced to a limit to support the wheel in the point area. If the wheel and frog are properly maintained, this design reduces the impact of the wheel in the open flangeway frog point area. Gradual ramping of the flangeway is critical to avoiding impact noise. A vertical spiral might alleviate impact noise as the wheel flange contacts the ramp. However, the flange height and flangeway depth must be carefully controlled to allow a spiral to work properly. Flange-bearing frogs are usually used in low-speed applications, such as crossovers.

Some longitudinal slip and torsion of the wheel set must occur as the wheel rolls over the flange-bearing portion due to the increased diameter of the flange relative to the tire. This is less of a problem at 90-degree diamonds, although some instantaneous acceleration and deceleration occur as the flanges pass over the flange-bearing portion. These effects contribute to wear of the flange and flange-bearing portion of the flangeway.

9.2.2.3 Lift Over Frog

Lift over frogs with welded toe and heel joints are similar to the flange-bearing design except the frog provides a continuous main line running rail surface and open flangeway. The lateral move flangeway is omitted in this design.

When a movement occurs for the diverging route, the frog flangeway and wing rail portion is ramped up to a level that allows the wheel to pass over the main line open flangeway and running rail head. If the wheel and frog are properly maintained, this design eliminates impact on the main line moves and reduces the impact of the wheel in the diverging direction.

The three frog designs described above are recommended for light rail transit installations to reduce noise and vibration. The frogs can be considered for three track types: ballasted, direct fixation, and embedded special trackwork.

9.2.2.4 Rail-Bound Manganese Frogs

Rail-bound manganese frogs with the running rail surrounding the central manganese portion of the frog introduce interface openings in the running rail surface in addition to the flangeway openings. Light rail main line track installations should always consider welded joints at the toe and heel of the frog. The manganese-to-rail-steel interface in the frog design introduces a joint in the running surface that severely impacts the wheel and is the source of wheel batter noise and vibrations from the outset of installation. They are not as quiet as the frogs described above.

9.2.2.5 Movable Point Frogs

Movable point frogs are perhaps the most effective of the various frogs for elimination of impact noise associated with fixed flangeway gap frogs. The frog flangeway is eliminated by laterally moving the nose of the frog in the direction in which the train is traveling. The movable point frog generally requires additional signaling, switch control circuits, and an additional switch machine to move the point of the frog. Movable point frogs have been incorporated on people mover systems in Canada and in Australia, but have received little application on light rail transit systems in the United States.

9.2.2.6 Spring Frogs

Spring frogs also eliminate the impact noise associated with fixed flangeway gap frogs for trains traversing the frog in a normal tangent direction. The spring frog includes a spring-loaded point, which maintains the continuity of the rail's running surface for normal tangent operations. For diverging movements, the normally closed frog is pushed open by the wheel flange. Additional noise associated with trains making diverging movements may occur because the train wheels must still pass through the fixed portion of the frog. Thus, use of these frogs in noise-sensitive areas where a significant number of diverging movements will occur will not significantly mitigate the noise impacts associated with standard frogs.

9.2.3 Curving Noise

Curving noise includes wheel squeal and flanging noise. Curving noise is one of the most serious types of noise produced by light rail transit systems and can occur at both short- and long-radius curves. In a central business district, pedestrians and patrons are in close proximity to embedded track curves, and, consequently, they are exposed to potentially high levels of squeal noise. The high level noise at discrete squeal frequencies is easily perceptible and annoying.

Curving noise may be intermittent due to varying contact surface properties, surface contaminants, or curving dynamics of the vehicle and rail. On wet days, wheel squeal may be eliminated when negotiating all or most of a curve.

9.2.3.1 Types of Curving Noise

The three assumed types of vibratory motion producing curving noise are the following:

- Longitudinal slip with non-linear rotational oscillation of the tire about its axle.
- Lateral slip with non-linear lateral oscillation of the tire across the rail head.
- Wheel flanging involving contact with the gauge face of the rail and tire slip across the rail head.

9.2.3.1.1 Longitudinal Slip

Longitudinal slip occurs on curves where the distance traversed at the high rail is greater than at the low rail. Wheel taper is sufficient to compensate for differential slip on curves with radii in excess of about 2,000 ft [610 m], although shorter radii may be accommodated by profile grinding of the rail head and gauge widening (which has undesirable effects). Further, Rudd reports that elastic compression of the inner wheel and extension of the outer wheel tread under torque can compensate for the wheel differential velocities, thus reducing the propensity for longitudinal slip.[49] Rudd further notes that trucks with independently driven wheels also squeal. The consensus of opinion is that longitudinal slip is not a cause of wheel squeal. However, it may cause a low-frequency rubbing sound if the slip is oscillatory.

9.2.3.1.2 Lateral Slip

Wheel squeal is sustained, saturated, non-linear transverse oscillation of the tire tread due to lateral creep or stick-slip of the tire across the rail head caused by the finite angle of attack (AOT) of the tire and rail.

Curving Geometry

Figure 9.2.12 illustrates the geometry of curve negotiation by a transit vehicle truck. Lateral slip across the rail head is necessitated by the finite wheel base (B) of the truck and the radius of curvature of the rail, where no longitudinal flexibility exists in the axle suspension. However, **Figure 9.2.13** illustrates the actual crabbing of a truck. In this case, the leading axle of the truck rides towards the high rail, limited only by flange contact of the high rail wheel against the gauge face of the rail. The trailing axle travels between the high and low rail, and the low rail wheel flange might, in fact, be in contact with the low rail gauge face. Gauge widening, common on many transit systems, increases the actual creep angle (angle of attack) and exacerbates the generation of wheel squeal. For additional information on truck rotation and behavior in curves, refer to Chapter 4, Articles 4.2 and 4.3.

Wheel squeal occurs most often at the low rail leading wheel, where flange contact with the rail gauge face does not normally occur. If a low rail restraining rail pulls the high rail wheel away from the high rail gauge face, lateral creep and wheel squeal can occur at the high rail wheel as well. For this reason, the rubbing of the tire against the restraining rail may be mistakenly blamed for causing wheel squeal.

The friction between the wheel and rail running surfaces during lateral slip varies non-linearly with the lateral creep, defined as the lateral slip velocity divided by the forward rolling velocity. The coefficient of friction initially increases with increasing creep rate, reaching its maximum at a creep rate of about 0.09, and declining thereafter. The negative slope describes a negative

damping effect that will produce regenerative oscillation or squeal if the damping is sufficient to overcome the internal damping of the system.

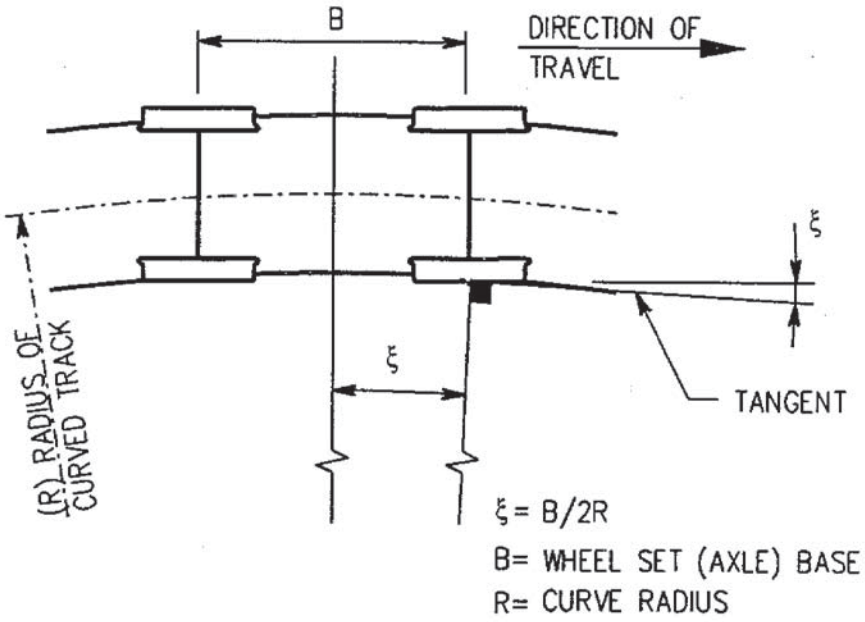

Figure 9.2.12 Geometry of curve negotiation and lateral slip

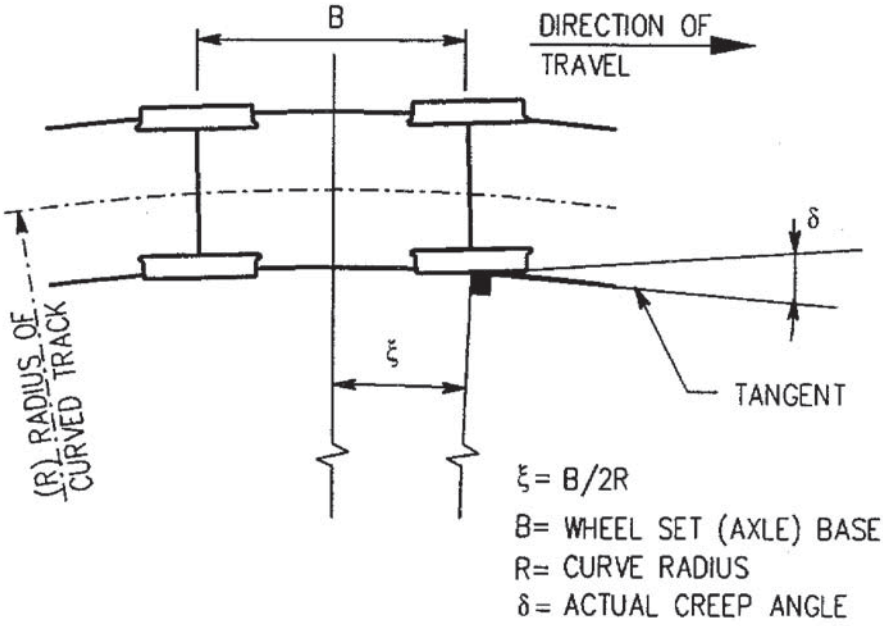

Figure 9.2.13 Truck crabbing under actual conditions

Squeal would not be expected for curve radii greater than 410 to 830 ft [125 to 253 m] and a wheelbase of 7.5 ft [2,280 mm], the lower limit being approached when there is no gauge widening. As illustrated above, gauge widening allows the creep angle to increase. A typical

assumption is that squeal does not occur for curves with radii greater than about 700 ft [200 m], corresponding to a dimensionless creep rate equal to 0.7 *B/R*, where *B* is the wheelbase and *R* is the curve radius.

Low-Floor Vehicles

Recently, the performance of "low-floor" transit vehicles with independently rotating wheels on center trucks and integral non-rotating center body sections has been studied by Griffen.[50] These vehicles have a reputation for higher noise levels during curving than those with conventional motored trucks and wheel sets, due to non-steering center trucks. The wheels of the center trucks rotate independently of one another, as they are not connected by solid axles. Thus, the self-steering feature of tapered wheels will not cause the high rail wheel to travel faster through the curve than the low rail wheel. Flanging noise during curving at curves with radii as large as 400 ft [122 m] can be substantial, requiring lubrication to control both noise and wear.

Meteorological Conditions

Meteorological conditions may affect the generation of squeal. In wet weather, for example, wheel squeal is greatly reduced due to the change in friction characteristics caused by moisture. Wheel squeal may be naturally inhibited in areas of high humidity.[51]

Tire Resonances

Wheel squeal involves distortional vibration of the tire, usually at frequencies above 1,000 Hz. Wheel squeal can be inhibited by damping treatment of the tire, such as by vibration absorbers or ring dampers, which damp the tire's distortional mode of vibration. The damping must be sufficient to overcome the negative damping associated with the friction-creep curve.

Figure 9.2.14 illustrates the lateral acceleration response of the TriMet Type II Vehicle resilient wheel, with and without vibration absorbers. (This response corresponds to the radial response shown in Figure 9.2.6.) The tire's lateral rigid body mode is at a frequency of less than 100 Hz, and the first distortional mode is again at about 500 Hz, followed by modes at about 1,400 and 2,600 Hz. All these resonances cause a very complex response that greatly complicates the analysis of wheel/rail interaction at audible frequencies.[52] In particular, curving noise will exhibit frequency components that are related to the lateral tire resonances shown in Figure 9.2.14. To the extent that vibration absorbers reduce the resonant response of the tire as shown in the figure, a reduction of wheel squeal might be expected. However, this is a non-linear oscillation, and if the positive damping provided by the absorber does not compensate sufficiently for the negative damping related to friction-creep, little reduction may be achieved. The resulting squeal may become saturated, limited only by the non-linearity of the tire's elastic response. From the track designer's perspective, controlling lateral slip by rail profile design, gauge, and superelevation is important.

Usually, resilient wheels such as those used at TriMet and other systems do not produce wheel squeal at curves, possibly due to the damping provided by the elastomer elements. The tire plane may also rotate relative to the axle, such that the conditions for wheel squeal are not sustained long enough to produce squeal. This does not prevent flanging noise, as discussed below. Further, resilient wheels do produce sustained wheel squeal, although usually for short

periods of time on the order of a second or two. This may be contrasted with solid wheels, where the sustained squeal may occur throughout curve negotiation.

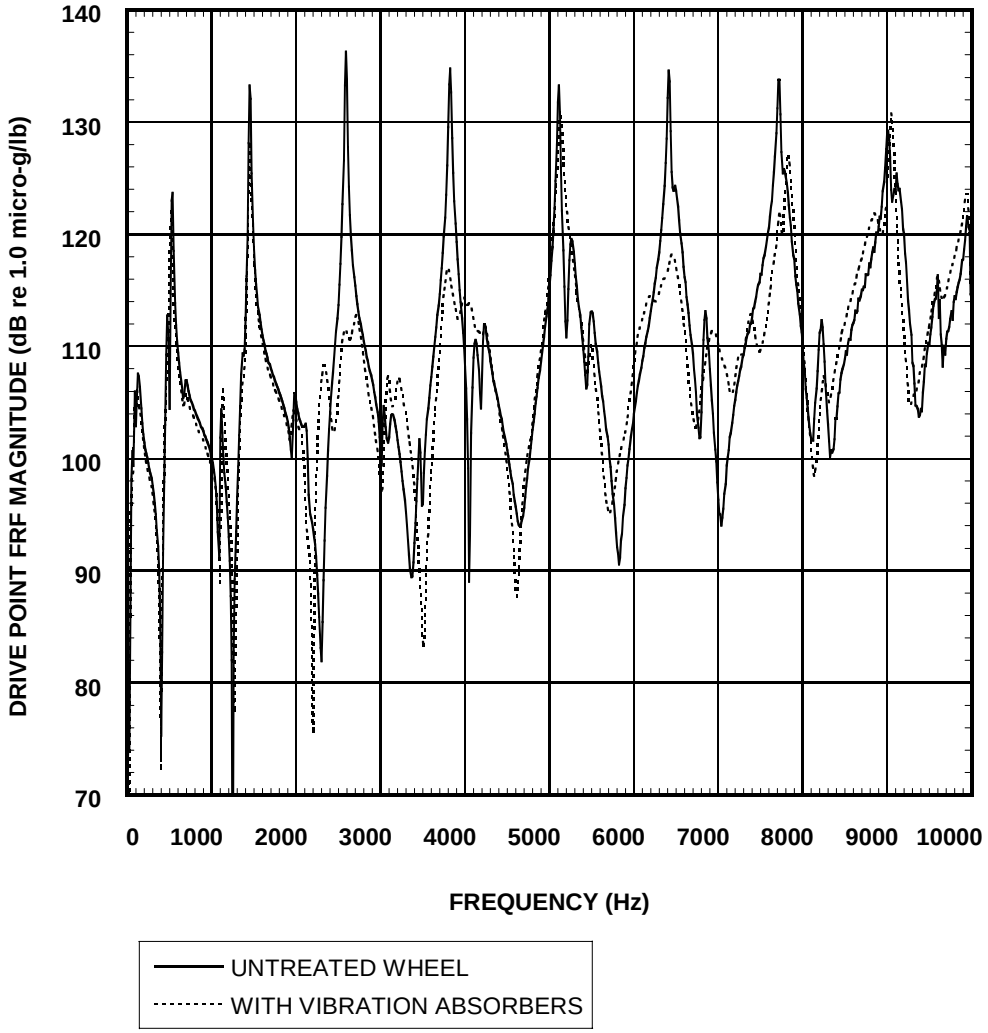

Figure 9.2.14 Transverse acceleration response of TriMet Type II resilient wheel

9.2.3.1.3 Flanging and Flanging Noise

Flanging involves a combination of the flange rubbing against the high rail gauge face and lateral slip of the tire across the rail head. Wheel flange rubbing against the high rail occurs on short-radius curves with significant crabbing of the wheel set, which may be exacerbated at gauge-widened curves, as illustrated in Figure 9.2.13. Stick-slip noise is generated during this process. However, the mechanism differs from that of sustained wheel squeal. In the case of flanging, the flange runs up against the gauge face, and the transverse reaction force against the flange and friction force between the tire and rail top surface induces a couple about a vertical axis through the center of the wheel. Eventually, as the wheel continues to roll, the tire rotates about the vertical until the friction is overcome, inducing a short chirp, again due to non-linearity of the

friction-creep curve and negative damping. The returning rotation and resulting slip ceases as soon as the tire reaches an equilibrium position, at which point the flange again approaches and contacts the gauge face. This process is repeated in an irregular fashion, producing intermittent squeal or chirping that can be objectionable, although it is usually not of the same magnitude as sustained wheel squeal. One may think of the wheel as "waddling down the rail." Flanging noise is very apparent with vehicles using resilient wheels, perhaps because the sustained wheel squeal does not usually occur to mask the flanging noise or perhaps because the resilient wheel tire is allowed to rotate about the vertical.

Flange rubbing against the gauge face does not clearly produce wheel squeal or chirping. However, considerable wear of the gauge face and flange occurs, and a high angle of attack promotes the wear process. As the gauge face and wheel flanges wear, the angle of attack can increase, thus exacerbating gauge face and wheel flange wear. As a result, flanging noise can increase.

9.2.3.2 Treatments for Curving Noise

A number of mitigation measures are available for controlling wheel squeal. The most effective of these from a track design point of view is lubrication. Resilient and damped wheels are not a component of track design, but their use greatly reduces the need for track or wayside noise control provisions. As with rolling noise control, curving noise control is a system problem rather than simply a vehicle or track design problem, and the track design engineer should be aware of various vehicle-based noise control provisions.

Treatments that may be incorporated in track design are discussed below.

9.2.3.2.1 Flange Lubrication

Wayside flange lubricators lubricate the rail gauge face, restraining rail, and wheel flange. The effectiveness of this type of lubrication in reducing noise can be substantial. Without lubrication, maximum wheel squeal noise levels may exceed 100 dBA. With lubrication, wheel squeal noise levels have been reduced by approximately 15 to 25 dB. The wheel squeal reduction is achieved by migration of a small amount of lubricant onto the top of the rail head. Back of flange lubrication necessarily transfers lubrication to the restraining rail.

However, while the lubricant tends to migrate to the running rail head, thus reducing wheel squeal due to lateral slip, traction loss occurs. The wheel tread and rail running surfaces cannot be lubricated without loss of adhesion and braking effectiveness. Loss of braking effectiveness will result in wheel flatting, which produces excessive rolling noise, a counterproductive result of improper lubrication. Erratic wheel-to-rail electrical contact from the use of uncontrolled wayside lubricants is also a concern. Environmental degradation by lubricants is a serious consideration; thus lubricants should be biodegradable to the maximum extent possible.

Current lubricants include (1) conventional petroleum products, (2) vegetable-oil-based grease, (3) Teflon-based lubricants, and (4) friction modifiers. Portland's TriMet has experimented with all forms of lubricants and has, evidently, as of this writing, settled on the Teflon-based lubricant as the most practical. Anecdotal evidence also indicates that the San Diego light rail system employs Teflon lubricants with flange lubricators with good success, achieving considerable carry distance on the order of miles.

9.2.3.2.2 Top-of-Rail Lubrication

Top-of-rail lubricators control wheel squeal by applying a controlled amount of lubricant to the top of the rail. Early versions of this included a conventional wiping bar with a canvas flap that directs the lubricant to the top of the rail. Recent designs include holes drilled through the rail head to allow lubricant to be pumped directly to the running contact surface of the rail.

9.2.3.2.3 Friction Modifiers

Friction modifiers applied to the rail head improve adhesion and flatten the friction/creep curve, thereby reducing or eliminating negative damping and wheel squeal while enhancing traction and braking. A problem associated with friction modifiers is clogging of distribution piping, as the modifier is not entirely a viscous fluid. Current technologies may have resolved this problem. (Friction modifiers are also applied directly to the wheel tread with “dry sticks” loaded directly from spring-loaded magazines.)

9.2.3.2.4 Water Sprays

Water sprays on curved track are used to control wheel squeal and flanging noise at some systems. Both the high and low rails can be treated. Water sprays may induce corrosion that is not conducive to electrical contact and may not be advisable for lightly used track or track where signaling may be affected. Water sprays would likely pose less of an environmental problem than grease or oil, but cannot be used during freezing weather.

9.2.3.2.5 Rail Head Inlays

The friction versus creep curve can be modified by treatment of the rail heads with a babbit-like (soft malleable metal) material. This treatment has eliminated wheel squeal, reducing noise levels by approximately 20 dB. However, after several months of service, “chronic squeal reappeared.” The loss of performance was likely due to wear of the material, allowing wheel tread contact with the native rail steel. Refer to Chapter 5, Article 5.2.5, for additional information concerning rail head treatments. Stainless steel inlays are used for improving electrical conductivity, a treatment which may affect wheel squeal generation. However, to the extent that wheel squeal involves lateral slip, any inlay would be expected to wear away.

9.2.3.2.6 Track Gauge

Gauge narrowing promotes curving and reduces angle of attack, lateral creep, and flanging noise. However, the wheel and rail gauges used on trolley systems typically vary by ⅛ inch [3 mm], and this slight variation in gauge may dictate against gauge narrowing in curves to prevent the flanges from binding when axle spacing is taken into consideration. Refer to Chapter 4, Article 4.2.4 for additional information concerning track and wheel gauge, specifically the use of Nytram plots to evaluate wheel flange clearances. Gauge narrowing by approximately ⅛ inch [3 mm] was employed at TriMet’s short-radius curves of radii 90 ft to 100 ft [27.5 m to 30.5 m] to control flanging noise and squeal.

Gauge widening (perhaps combined with restraining rail) has been incorporated in track design in an attempt to control squeal and promote curving, but this has produced the opposite effect. Gauge widening appears to be a holdover from steam locomotive days when three-axle trucks were in use. Gauge widening is not specifically necessary to prevent excessive flange wear for two-axle trucks. Quite the opposite, gauge widening promotes crabbing because the natural tendency of a truck is to crab its way through a curve, with the high rail wheel flange of the

leading axle riding against the high rail gauge face, as illustrated in Figure 9.2.13. Gauge widening should not be used to control curving noise.

9.2.3.2.7 Asymmetrical Rail Profile

Asymmetrical rail head profiles are designed to increase the wheel rolling radius differential and promote self-steering of the truck through the curve, which requires a longitudinally flexible truck suspension. In this case, the contact zone of the high rail is moved toward the gauge corner and the larger diameter of the tapered wheel, while the contact zone at the low rail is moved to the field side and the smaller diameter of the tapered wheel. The wheel taper thus allows the high rail wheel to travel a greater distance than the low rail wheel per revolution without slip. In so doing, the axles tend to line up with the curve radius, thus reducing the angle of attack and lateral slip. While this approach is attractive, it is only effective for curve radii on the order of 700 ft [200 m] or more and is less effective with primary suspensions with very high longitudinal stiffness. This provision has been used in Los Angeles and Vancouver.

9.2.3.2.8 Rail Vibration Dampers

Rail vibration dampers are described in Article 9.2.1.3.8. A rail vibration damper is a viscoelastic constrained layer damping system applied to the rail web. The constrained layer is held against the rail web with a steel plate and spring clip under and about the base of the rail. The absorbers can be applied with minimal disturbance of track, provided that they are short enough to fit between the rail supports. Rail vibration absorbers are not specifically expected to eliminate wheel squeal, as most of the vibration strain energy is in the wheel tire. They might reduce some of the rail vibration energy to the extent that it involves resonances in the rail. Tests were conducted at the MBTA Green Line's short-radius curve at Government Center with limited success.[52]

A second design includes a damping compound that is bonded to the rail web and constraining steel plate, without the use of a steel spring clip. The performance of this type of absorber may be of limited effectiveness in controlling wheel squeal, although some rail-radiated noise reduction might be obtained.

9.2.3.2.9 Tuned Rail Vibration Absorbers

Tuned rail vibration absorbers are discussed in Article 9.2.1.3.7 with respect to rolling noise control. Rail vibration absorbers are reputed to control wheel squeal and also reduce rolling noise. The most attractive design at present incorporates a series of tuned dampers that bear against both the rail foot and the rail web. Thus, vibration energy is absorbed from both of these elements of the rail. The absorbers are clamped to the rail with bolts, and a plate extends beneath the base of the rail. These systems have been used in Europe, but not in North America. This technology has been tested at TriMet at both tangent and curved track.[52] While tuned vibration absorbers may give favorable qualitative performance with respect to rolling noise, their ability to control wheel flanging noise where resilient wheels are used is limited at best.

9.2.3.2.10 Double Restrained Curves

Double restraining rails are designed to reduce the angle of attack and promote steering of the truck without flange contact on gauge-widened curves. In this case, the leading high rail wheel flange can be brought away from the high rail by the low rail, or inner, restraining rail, and the trailing low rail wheel flange can be moved away from the low rail gauge face by the high rail restraining rail, thus reducing angle of attack and lateral creep. Longitudinal slip would also be

reduced with conical tread profiles, provided that wheels are trued with adequate frequency. A detailed analysis of curving and flangeway clearances must be done if success is to be obtained. The restraining rails would have to be carefully adjusted to optimize performance, but wear over time may reduce or eliminate any effectiveness that they might have to offer. The restraining rail flangeway width would have to be controlled to prevent binding of the wheel set or climbing of the flange onto the restraining rail. Wear of the restraining rail may require frequent adjustment, thus leading to increased maintenance cost.

The restraining rails should be liberally lubricated to reduce squeal noise and wear due to friction between the wheel and restraining rail. Lateral slip of the wheels will still occur, as illustrated in Figure 9.2.12, so top-of-rail lubrication may be required to prevent squeal, although this might be achieved by migration of gauge face lubricant to the top of rail.

Although this approach is theoretically attractive in reducing crab angle, mixed results may be achieved. Examples can be cited where double restrained curves did not prevent sustained wheel squeal, such as the Boston Blue Line downtown turn-around curve. The double restrained curve at the Portland TriMet Sylvan Hills East Portal did not prevent wheel squeal and flanging noise. Thus, double restraining rails for controlling wheel squeal are not specifically recommended for noise control. Refer to Chapter 4, Article 4.3 for additional information concerning guarded track and restraining rail.

9.2.3.2.11 Low Rail Cant

Researchers at TNO in the Netherlands have produced experiments suggesting that canting the low rail towards the field side may allow a negative feedback effect that inhibits stick-slip of the leading low rail tire across the rail head, thus eliminating squeal. Rail freight vehicles have been observed to traverse curves with jointed ballast and wood tie track with tie plates and cut spikes without squealing, while similar vehicles on newly installed continuous welded rail with concrete ties, spring clips, and tie pads, produced substantial squeal. The wood tie track might have allowed the low rail to roll toward the field side, thereby inhibiting stick-slip oscillation of the low rail leading tire. The efficacy of using rail cant to reduce squeal at transit is unknown, but is mentioned here as a possible design concept that may be explored. Additional research is required.

9.3 VIBRATION CONTROL

Ground-borne noise and vibration are phenomena of all rail transit systems that, if not controlled, can significantly impact residences, hospitals, concert halls, museums, recording studios, and other sensitive land uses. New light rail transit alignments include abandoned railroad rights-of-way passing through adjacent residential developments. Residences located within 3 ft [1 m] of the right-of-way limits are not uncommon, and there are instances where apartment buildings are built directly over light rail systems with little provision for vibration isolation. Vibration impacts on hospitals, sensitive “high-tech” manufacturing facilities, or research facilities may occur. Transit-oriented developments (TOD) are attractive, in that they combine commercial and residential structures with transit station structures. In this case, control of structure-borne noise is critical. The most cost-effective mitigation measure for transit-oriented development is to provide a vibration-isolated track, right at the source.

Ground-borne noise is heard as a low level rumble and may adversely impact residences, hospitals, concert halls, and other areas or land uses where quiet is either desirable or required. Ground-borne vibration in buildings may be felt as a low-frequency floor motion or detected as secondary noise such as rattling windows or dishes. Building owners often claim that ground-borne vibration is responsible for building settlement and damage, although no demonstrated cases of this occurring have been verified.

Literature concerning rail transit ground-borne noise and vibration control is rich with empirical and theoretical studies conducted in North America, Europe, Australia, the Far East, and South America. A substantial review of the state-of-the-art in ground-borne noise and vibration prediction and control was conducted in 1984 for the U.S. Department of Transportation.[53] Recent research includes studies on the nature of subway/soil interaction, surface track vibration generation, and extensive down-hole testing to assess vibration propagation in soils. The prediction of ground-borne noise and vibration has advanced to a highly developed state, relying on shear wave velocity and seismic refraction data, borehole impulse testing, seismic modeling, and detailed finite element modeling of structures and surrounding soils. As a result, vibration predictions can be reasonably accurate. Special track isolation designs are now regularly considered as a means to control perceptible ground vibration in addition to audible ground-borne noise.

9.3.1 Vibration Generation

Ground vibration from rail transit vehicles is produced by wheel/rail interaction, driven by roughness in the wheels and rail running surfaces, rail undulation, discrete track structures, track irregularities, and imbalance of rotating components such as wheels and axles. The spectrum of low-frequency ground vibration is affected strongly by axle spacing, truck spacing, and wheel diameter. Vibration forces are imparted to the track invert or soil surface through embedded track, direct fixation fasteners, or ballast.

These forces cause the transit structure and soil to vibrate, radiating vibration energy away from the track in the form of body and surface waves. Body waves are shear and compression waves, with respective shear and compression wave propagation velocities. Body waves attenuate (or lose amplitude) at a rate of roughly 6 dB (50% in amplitude) as the distance from a point source doubles without material damping (energy absorption) in the soil. The rate of attenuation is 3 dB per doubling of distance from a line source such as a train. Of these two types of waves, the shear wave is the most important. For surface track, the ground vibration includes Rayleigh surface waves that attenuate at a rate of 3 dB (30% in amplitude) as distance from the point source doubles without material damping or reflection from lower soil layers. The rate of attenuation of a Rayleigh surface wave from a line source is nil without material damping. Fortunately, surface soils have considerable material damping. Rayleigh surface waves are the major carrier of vibration energy from the surface track, but non-homogeneities in the soil may convert significant portions of the Rayleigh surface wave energy into body waves. Within one wavelength of the track, near-field responses dominate the response.

Structure/soil interaction significantly affects the radiation of vibration energy into the surrounding soil. Heavy tunnel structures produce lower levels of ground vibration than lightweight tunnels, depending on soil stiffness and frequency of excitation. However, the opposite has been observed

for large cut-and-cover box structures very close to the ground surface relative to circular tunnels. Near-surface subway structures produce vibration more easily than deep structures.

Ground vibration excites building foundations and structures. Vibrating surfaces of the rooms then radiate noise into the room as ground-borne noise. The interior sound level is then controlled by the degree of acoustical absorption contained in the room. Secondary noise, such as rattling windows, might be observed in extreme cases.

9.3.2 Ground-Borne Noise and Vibration Prediction

A number of procedures are available to determine needed ground-borne noise and vibration control provisions. The main procedure is an empirical approach involving transfer function testing of soils and buildings. The procedure has been adopted by the FTA and FRA for assessing ground-borne noise and vibration impacts by rail transit and high-speed rail projects, respectively. The predictions of ground vibration and ground-borne noise are described in detail in the FTA guidelines for rail transit noise and vibration impact assessment.[54] Screening procedures and detailed prediction techniques are also described.

The state-of-the-art in predicting ground vibration has recently advanced significantly to include detailed finite element modeling of soil/structure interaction,[55] numerical analysis of vibration propagation in layered soils using both analytical and finite element modeling methods, and multiple-degree-of-freedom modeling of transit vehicles and track.[56] These methods are very powerful for analyzing changes in structure design, structure depth, and vehicle designs.

Ground vibration has been predicted to long ranges on the order of 3,000 ft [1,000 m] by seismic modeling for the Sound Transit system tunnels proposed for the University of Washington Campus. Vibratory compactors have been employed for measuring long-range responses on the campus. Thus, a variety of prediction tools have been developed.

9.3.3 Vibration Control Provisions

Numerous methods for controlling ground-borne noise and vibration include continuous floating slab track, resiliently supported two-block ties, ballast mats, tire-derived aggregate (TDA), resilient direct fixation fasteners, precision rail, alignment modification, low-stiffness vehicle primary suspension systems, and transmission path modification.[57] Achieving the most practical solution at reasonable cost is of great importance in vibration mitigation design. Factors to consider include maintainability, ease of inspection, and cleanliness.

9.3.3.1 Floating Slab Track

Floating slab track is a special type of track structure that is beyond the normal designs discussed in Chapter 4. The floating slab concept would be an additional requirement to normal track structure. Track structure design must allow for floating slabs where they are needed, as the floating slab usually requires additional invert depth.

Floating slab systems consist of two basic types:

- Continuous cast-in-place floating slabs are constructed by placing a permanent sheet metal form on elastomer isolators and filling the form with concrete. The floating slabs measure

approximately 20 ft [6 m] or more along the track and 10 ft [3 m] transverse to the track. The depth of the slab is generally 12 to 18 inches [300 to 450 mm]. Some construction techniques involve using the tunnel invert as a form for the slab, then jacking the slab to design elevation.

- Discontinuous "double tie" precast floating slabs measure about 4 to 5 feet [1.2 to 1.5 m] along the track and 10 ft [3 m] transverse to the track. The depth, and thus the mass, of the slab may vary from about 8 to 24 inches [200 to 600 mm]. The weight of the slab may range from 4,400 to 15,000 lb [2,000 to 7,000 Kg]. The most common configuration is with a 4,400 lb [1,800 Kg] slab that is 8 inches [200 mm] thick. The slabs are referred to as double ties because they support each rail with two direct fixation fasteners, giving a total of four direct fixation fasteners per slab.

The design resonance frequency of a floating slab system is the resonance frequency for the combined floating slab and vehicle truck mass distributed over the length of the vehicle. The design resonance frequency of the continuous floating slab and vehicle combination is typically on the order of 16 Hz, while that of the discontinuous precast double tie floating slab and vehicle combination ranges from 8 to 16 Hz, depending on isolation needs. With a continuous floating slab, the entrained air stiffness must be included with the isolator spring stiffness when computing the resonance frequency.

The normal configuration for the discrete double tie design includes four natural rubber isolators. Additional isolators are incorporated to increase the isolation stiffness at transition regions between non-isolated and isolated track. The main support pad design should provide low shear strain and control lateral slip between the bearing surface of the pad and concrete surfaces. Lateral slip is further reduced by gluing the pads to the concrete surfaces. The typical main support pad is about 3 to 4 inches [75 to 100 mm] thick, with an overall diameter of 12 to 16 inches [300 to 400 mm].

The isolators used at almost all floating slabs in the United States are manufactured from natural rubber. Synthetic rubber formulations exhibit higher creep rates than natural rubber formulations. Natural rubber formulations exhibit low creep over time, high reliability, and dimensional stability. Natural rubber pads are not subject to corrosion and provide natural material damping that controls the amplification of vibration at resonance. Natural rubber pads give a virtually maintenance-free isolation system.

Steel coil springs are used for many continuous poured-in-place floating slabs in Europe and the Far East. Steel coil springs provide a low ratio of dynamic-to-static stiffness, essentially equal to unity. This can be an advantage where a low resonance frequency system with high attenuation is needed, but static deflection must be kept to a minimum. The disadvantages of steel coil spring isolators include low damping with potentially high amplification at resonance, surge at audible frequencies, and corrosion. Neoprene noise pads and dampers are usually employed to control spring surge. Corrosion-inhibiting coatings are applied to the spring's metal components. Provisions are made in the slab design to allow easy removal and replacement of springs that might be corroded.

In North America, a steel coil spring floating slab track was installed in Charlotte, North Carolina. Numerous steel spring systems have been installed in Europe and the Far East, some providing exceptionally low isolation frequency and high attenuation.

Concerns exist regarding debris accumulating beneath floating slabs and how to remove such debris. Another concern is the possibility of the gaps between discontinuous floating slabs trapping the feet of persons escaping down a tunnel during an emergency. Both of these concerns may be avoided by providing flexible seals. Access holes or cut-outs in the slab can provide access for debris removal.

Low-frequency floating slabs have been developed for controlling structure-borne noise at combined residential and transit station structures and controlling low-frequency ground vibration impacts on sensitive research and manufacturing facilities. In the former case, the slab isolation frequency is about 8 Hz. In the latter case, the isolation frequency may be on the order of 5 Hz. Low-frequency continuous poured-in-place floating slabs with steel coil spring isolators, developed by GERB, have been installed in a number of systems around the world, including the installation at Charlotte, North Carolina, on the Charlotte Area Transit System (CATS) LYNX Blue Line. Low-frequency discontinuous double tie floating slabs with natural rubber isolators have been designed for the North Link tunnels at Sound Transit to control low-frequency ground vibration at long-range receivers on the University of Washington Campus, the MARTA Northside Extension to control medical building vibration, and the Chatswood Interchange in Chatswood, New South Wales (Sydney). This latter project involved vibration isolation of commuter trains from a station structure that would also support a high-rise condominium tower.

9.3.3.2 Resiliently Supported Bi-Block Ties

Resiliently supported bi-block tie designs are referred to as encased direct fixation track in Article 4.6.3.3. With resiliently supported bi-block tie designs, each rail is supported on individual concrete blocks set in an elastomer boot encased by the concrete slab or invert. A stiff elastomer or plastic rail seat pad protects the concrete block at the rail base, which is retained by a spring clip or other fastening system. The main advantage of the bi-block tie system is that anchor bolts are not required, thus reducing part count and improving reliability.

The design used for light rail transit vibration isolation must provide a low rail support modulus, achieved by including a closed-cell elastomer foam (or micro-cellular pad) between the bottom of the concrete block and invert inside the elastomer boot. A static stiffness on the order of 100,000 lb/in [18 KN/mm] can be obtained, although the dynamic stiffness is likely to be much higher. The high-frequency (above 200 Hz) vibration isolation provided by resiliently supported two-block ties is believed to be higher than that of very stiff direct fixation fasteners. The low-frequency vibration isolation provided by the two-block tie should be comparable to that provided by soft fasteners. Damping has been postulated as a cause for the low-frequency vibration isolation provided by some of the two-block systems.

Rail corrugation associated with the resiliently supported tie system has been reported, although this appears to be related to the interaction of the rail with the concrete block through the rail seat pad. The design constitutes a two-degree-of-freedom vibration isolation system. As a result, the rail and block can vibrate against each other, at high frequencies, thus possibly contributing to rail corrugation. Reducing the rail seat pad stiffness appears to defer the onset of rail corrugation. Rail pads that are too soft may induce clip fatigue, so clip design must be carefully reviewed. Other concerns include abrasion of the concrete block and production of fines that are trapped in the boot. Water can be trapped in the boot as well.

9.3.3.3 Ballast Mats

Ballast mats control ground-borne noise and vibration from ballasted track and have been incorporated as the principal isolation system. Two configurations of ballast mats have been employed for surface track. The first includes a concrete base or tub with a ballast mat consisting of inverted natural rubber cone springs placed on a concrete base beneath the ballast. The second, and potentially less effective, design incorporates a uniform ballast mat placed directly on tamped soil or compacted subballast. (Ballast mats have also been placed on a concrete "bath tub" slab with the track slab consisting of a second pour concrete slab supporting the rails.)

Conventional installations of ballast mats in European subways have concrete bases for which vibration insertion losses have been predicted to be higher than observed in practice. Surface track application presents challenges that limit the effectiveness of ballast mat installations. The shear modulus of the soil at or near the surface may be low and can offer a track support modulus comparable to that of the ballast mat, thus rendering the ballast mat less effective than if it were employed in a tunnel or concrete U-wall section.

The vibration reductions are limited to the frequency range in excess of about 30 Hz. For ballast mats on compacted subgrade, the insertion loss would likely be about 5 to 8 dB at 40 Hz. For ballast mats on a concrete base or concrete invert, the insertion loss at 40 Hz would be between 7 and 10 dB. The most effective ballast mat is a profiled mat with a natural rubber elastomer on a concrete base or trough. This type of installation provides the greatest vibration isolation, about 10 dB at 40 to 50 Hz. The ballast mat is too stiff to provide sufficient vibration reduction at lower frequencies and is, therefore, not a substitute for floating slab track. There may be some minimal amplification of vibration at the ballast mat resonance frequency in the range of 16 to 30 Hz.

The selection of a ballast mat should favor low static and dynamic stiffness, low creep, good drainage, and ease of installation. Considerable disparities exist between the dynamic stiffness of various ballast mats, even though their static stiffness may be similar. The most desirable material is natural rubber, which exhibits a low dynamic-to-static stiffness ratio of about 1.4 or less. These high-performance natural rubber mats may cost more than synthetic elastomer mats, but may be the only choice in critically sensitive locations. Specifications for ballast mats should include dynamic stiffness requirements for the intended frequency range over which vibration isolation is desired. If this is not done, much less isolation than expected may actually be achieved, rendering the vibration isolation provision ineffective and possibly detrimental. There is a very distinct possibility that providing a ballast mat may increase low-frequency vibration in the 16- to 25-Hz region by a few decibels. If this is the range of the most significant vibration, the ballast mat may actually exacerbate a vibration impact. Thus, great care must be exercised in design, specification, and installation of the ballast mat.

A further consideration is ballast pulverization and penetration into the mat. Ballast mats have been incorporated in the track structure to reduce pulverization.[53]

9.3.3.4 Tire-Derived Aggregate (TDA)

Tire-derived aggregate (TDA) is used for isolating ballasted track and consists essentially of shredded tires of particular sieve size. The tire-derived aggregate is attractive because it (1) is economical, (2) provides a use for waste tires, and (3) is easy to install. The typical installation consists of 12 inches [300 mm] of TDA wrapped in geo-textile, placed on compacted subgrade,

and covered with 12 inches [300 mm] of subballast and 12 inches [300 mm] of ballast, directly beneath the cross ties. The TDA is compacted in place. Variations in design may exist.

The vibration isolation effectiveness of TDA track is comparable or perhaps slightly better than that of the most effective ballast mats. **Figure 9.3.1** shows the measured vibration response of the ground near a ballasted track section of the VTA Line in San Jose, California, with TDA relative to without TDA underlayment.[58] The treatment installation was as described above. The treatment was effective above 25 Hz, providing as much as 10 dB reduction at 63 to 125 Hz. The aggregate was installed in 2005, and the measurement data with aggregate are for the years 2005, 2006, and 2009. Some loss of vibration isolation performance is appears to have occurred by year 2009 relative to years 2005 and 2006, perhaps due to further compaction or fines. However, this result is also within experimental error. No additional data have been collected.

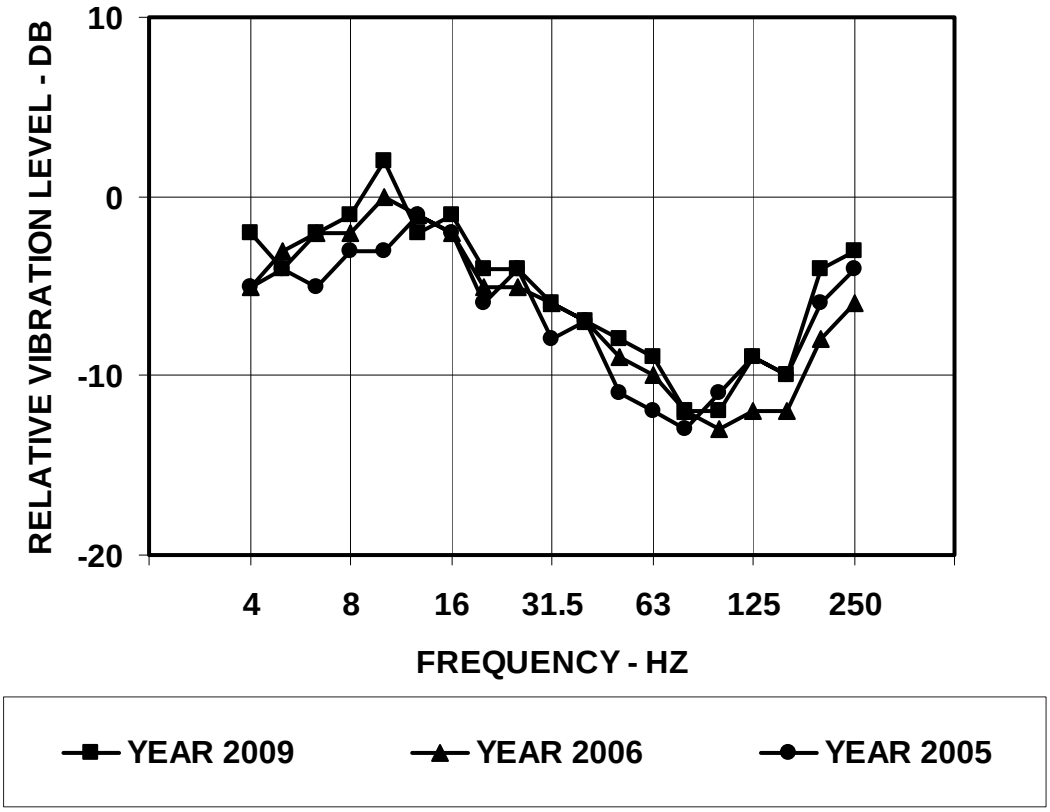

Figure 9.3.1 Vibration isolation performance of tire-derived aggregate installed in 2005

As of this writing, the FTA has not approved TDA as a vibration control measure for general use, but its use has been provisionally approved on the BART extension to San Jose. It has been

installed on the San Jose VTA system on the Vasona Line and also in Denver as part of the T-Rex project. A record of performance is being developed.

9.3.3.5 Resilient Direct Fixation Fastener Design for Vibration Isolation

Resilient direct fixation fasteners used for supporting the rail on concrete slabs or inverts are very common at heavy rail transit systems where subway and aerial structures are involved. Resilient direct fixation fasteners control structure-borne and ground-borne noise and vibration and can be provided with a wide range of stiffness values, allowing the designer to adjust rail support modulus as needed. In general, soft fasteners transmit less structure-borne or ground-borne noise and vibration than stiff fasteners do, whereas stiff fasteners reduce rail vibration and rail-radiated noise more than soft fasteners do. The selection of a fastener's stiffness will, to some extent, depend on the type of structure and the nature of the vibration or noise that is to be controlled.

The static stiffness ranges of fasteners are described as follows:

Soft:	50,000 to 80,000 lb/in [9 to 14.2 KN/mm]
Medium:	80,000 to 140,000 lb/in [14.2 to 25 KN/mm]
Stiff:	140,000 lb/in [25 KN/mm] or greater

Two basic types of fasteners are available. One type is a fastener with elastomer bonded to a base plate and top plate and is referred to as a bonded fastener. The second type of fastener consists of an elastomer sandwiched between a base plate and top plate without bonding. In some cases, the base plate may be absent, as with the original TTC resilient fastener employed on the YSNE tunnels. In other cases, the rail is supported directly by the elastomer.

The stiffness of the fastener is controlled by the durometer and geometry of the elastomer. Solid elastomer is essentially incompressible, so the compliance of an elastomer spring in compression is achieved by providing free surfaces that allow the elastomer to expand as the elastomer is deflected under load. The ratio of the area of one loaded surface to the area of the free surface is the "shape factor" of the elastomer spring. Elastomer springs with low shape factors are softer than elastomer springs with high shape factors. An elastomer spring with infinite shape factor is for most purposes considered infinitely stiff.

High-compliance fasteners with static stiffness less than 80,000 lb/in [14 KN/mm] use elastomer in shear to provide good rail head control with low vertical stiffness and can be effective in reducing ground vibration and ground-borne noise at frequencies above about 30 Hz. The elastomer-in-shear design can provide a vertical static stiffness as low as 50,000 lb/in [9 KN/mm] and can employ a captive top plate within the bottom plate, providing stability in the event of elastomer failure (which rarely, if ever, occurs). For additional information on direct fixation fasteners, refer to Chapter 5 in this Handbook.

Some non-bonded fasteners employ a closed-cell urethane cellular pad with a highly non-linear load deflection curve that is actually concave downward. One form of urethane is subject to water absorption and degradation. The type used for track isolation is supposedly resistant to water absorption. These elastomers do not require unloaded free surfaces (shape factor) to provide resilience. These unbounded fasteners required anchor bolts to pass through the top plate to retain the top plate and also laterally restrain the fastener. Springs provide

precompression. This type of fastener has not gained significant application in the United States, although the New York Transit Authority has evidently installed some of these.

A low-stiffness fastener will allow the rail to deflect over a larger distance under static load than a high-stiffness fastener. The axle load is distributed over more fasteners with low stiffness than over fasteners with high stiffness, reducing anchor bolt and plinth stresses. Low rail support stiffness is advantageous in reducing the vertical resonance frequency of the rail on the fastener stiffness and the so-called P2 resonance frequency.

High-compliance fasteners isolate the rail from concrete invert non-uniformity and, thus, reduce very-low-frequency vibration that might otherwise occur. This is beneficial in areas where low-frequency vibration might impact vibration-sensitive manufacturing facilities or where a subway or slab track is located in soft soil close to residences or other sensitive uses.

Dynamic-to-Static Stiffness Ratio

The ratio of vertical dynamic-to-static stiffness describes the stiffness of the fastener at frequencies associated with ground vibration and noise. A low ratio is generally associated with high-quality natural rubber elastomer and desirable for vibration isolation. The ratio is obtained by dividing the dynamic stiffness (measured with a servo-actuated hydraulic ram) by the static stiffness determined over the majority of the load range. The ratio is not entirely a material property of the elastomer, but is also a function of shape factor. A desirable upper limit is 1.4, easily obtained with fasteners manufactured with natural rubber or a derivative thereof. Dynamic-to-static stiffness ratios of 1.3 are not uncommon with natural rubber elastomer in shear. As a rule, elastomers capable of meeting the limit of 1.4 are high quality and generally exhibit low creep. Fasteners with neoprene elastomer usually have a dynamic-to-static stiffness ratio greater than 1.7 and as high as 4. As noted in Article 9.2.1.3.5, a neoprene elastomer may be desirable for controlling rail noise radiated from at-grade or aerial structure track due to the material damping of the elastomer. Thus, the choice of a specific type of elastomer may depend on whether ground-borne vibration isolation or airborne noise reduction is desired.

Stiffness Selection

The vibration isolation effectiveness of a rail fastening system is generally controlled by the dynamic rail support modulus, computed by dividing the fastener's dynamic stiffness by the rail support pitch, or center-to-center spacing. The vibration isolation performance of resilient direct fixation fasteners as a function of rail support modulus relative to a rail support modulus of 4,300 lb/in^2 [30 MN/m^2] assumed for the TTC standard unbounded fastener with single 45 Durometer natural rubber pad (circa 1970) was investigated by Bender.[59] The results are summarized in **Figure 9.3.2**. (The TTC pad may actually be considerably stiffer than represented here.) The model assumes an unsprung solid wheel set mass rolling on the rail, without consideration for the vehicle truck dynamics. These predictions are reasonably well supported by field tests conducted by the author over the years and are used for DF fastener stiffness selection. The maximum rail stresses and deflections due to bending as a function of rail support modulus are listed in **Table 9.3.1** for a 30,000 lb load [133 kN] point. [59]

The theoretical analysis employed for the predictions given in Figure 9.3.2 indicates that the net force transmitted by discrete resilient direct fixation fasteners over the frequencies indicated is

equivalent to that predicted by a single-degree-of-freedom isolator with rail mass per unit length supported by the rail support modulus. This simple result simplifies the problem of computing the transmitted force for continuous rail support. The assumption of continuous rail support is adequate for frequencies up to about half of the pinned-pinned mode frequency of the discretely supported rail, or about 300Hz.

A good dynamic rail support modulus for vibration isolation that is similar to that often assumed for ballast-and-tie track is 3,000 lb/in^2 [21 MN/m^2] or less. This implies a rail fastener dynamic stiffness of 90,000 lb/in [16 KN/mm] for a pitch of 30 inches [750 mm] or static stiffness of 65,000 lb/in [11.4 KN/mm]. Fasteners capable of providing this low stiffness incorporate natural rubber-in-shear. Bonded fasteners with elastomer in compression can also provide low stiffness if the elastomer is sufficiently thick, with sufficiently low shape factor.

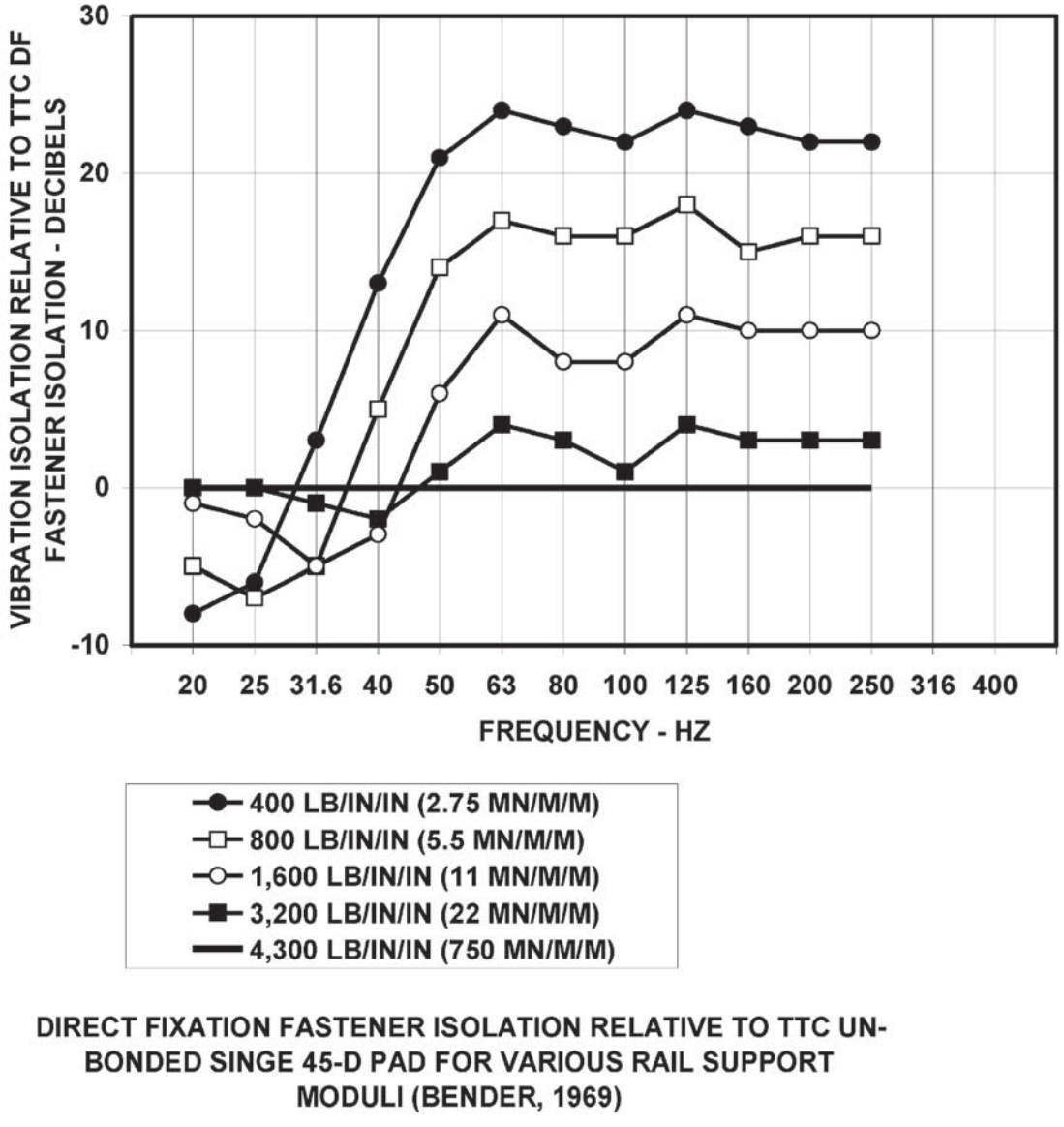

Figure 9.3.2 Vibration isolation of DF fasteners for various rail support moduli

Aerial Structures

The selection of the fasteners for aerial structure application should be based, among other things, on whether the aerial structure would be constructed with concrete or steel or a combination thereof. All-concrete aerial structures tend to radiate less structure-borne noise than that radiated by the rail and wheel. Steel structures radiate considerable low-frequency noise, and concrete deck and steel box girder aerial structures may produce low-frequency rumble. Thus, a stiff neoprene elastomer with greater loss factor than that of natural rubber may be preferable for a fully concrete aerial structure application, while a high-compliance fastener may be preferable for steel elevated structures or composite steel box elevated structures where maximal isolation is preferred.

An analysis of the relative contributions of the rail and structure to radiated noise was conducted for the NYCTA.[60] Testing at the NYCTA showed that softer fasteners performed better than stiffer fasteners in controlling noise radiation by steel elevated structures. The best performance was obtained with bonded resilient direct fixation fasteners with static stiffness of about 100,000 lb/in [17.5 KN/mm], a ratio of dynamic-to-static stiffness of less than 1.4, linear load/versus deflection characteristic, and top plate bending resonance in excess of 800 Hz.[61]

The tendency today in direct fixation track design is to provide fasteners with static stiffness on the order of 50,000 lb/in to 150,000 lb/in [9 to 27 MN/m], utilizing natural rubber elastomer or a blend of natural rubber and synthetic rubber. As noted above, while natural rubber has desirable properties for vibration isolation, the low damping capacity of these materials may allow efficient bending wave propagation and consequent noise radiation by the rail. Elastomeric fasteners with medium stiffness and damping, such as neoprene, are suitable for concrete aerial structures where rail-radiated noise is dominant. Natural rubber fasteners with low stiffness and low damping are suitable for tunnels to control ground-borne noise and vibration and for steel elevated structures or aerial structures with steel box or I-Beam girders to control structure-radiated noise.

Non-linear Load vs. Deflection

The load vs. deflection curve of the fastener should be linear within +/-15% of the mean static stiffness over the load range to avoid excessive stiffness while providing good rail support. Most of the vibration energy transmitted by a fastener to the invert is transmitted when the rail is directly over the fastener or within several feet. Fasteners with highly non-linear load versus deflection characteristic are much stiffer when the wheel is passing over the fastener than when supporting just the rail. The vibration forces transmitted by the stiffness are proportional to the dynamic stiffness of the fastener, which is typically 1.4 times the tangent to the load vs. deflection curve under the maximum operating load.

Specifying linearity in an unambiguous way is critical in the procurement process. The fastener should provide full three-degree-of-freedom isolation. Horizontal snubbing is sometimes achieved by incorporating a positive restraint between the top and bottom plate that can cause high non-linearity of the load deflection curve and compromise vibration isolation. Hard snubbers should be avoided in fasteners, because they limit vibration isolation to the vertical direction only. The fastener should have a finite lateral stiffness measured at the rail base to ensure both an adequate degree of horizontal position control and sufficient lateral compliance to provide three-

degree-of-freedom vibration isolation. Fasteners with elastomer in shear provide a linear load deflection curve and excellent vibration isolation performance over a wide range of loads.

Top Plate Design

Weakness in the top plate tends to reduce the stiffness of the fastener, but allows the top plate to resonate in bending between 500 and 1,000 Hz, amplifying transmitted forces at resonance. Many, many, other resonances occur, some of which may be important, and many which may not. The top plate bending is singled out because its frequency is typically within the range of audible wheel/rail noise and also within the range of short pitch corrugation frequencies.

Table 9.3.1 Maximum stress and deflection for 30,000 lb [133 kN] point load

RAIL SUPPORT MODULUS		MAXIMUM STRESS		DEFLECTION	
LB/IN^2	MN/M^2	LB/IN^2	MN/M^2	INCHES	MM
400	2.8	22,000	152	0.565	14.3
800	5.5	19,300	133	0.344	8.7
1,600	11	16,000	110	0.196	5.0
3,200	22	13,500	93	0.117	3.0
4,300	30	12,500	86	0.093	2.4

Although transmitted forces are amplified, the resonance produces a minimum in the mechanical input impedance of the top plate, as seen by the rail. At this frequency, the fastener looks like a dynamic absorber, absorbing vibration energy from the rail and eliminating singing rail at this frequency. See the discussion above regarding tuning of the top plate and also the ¼-wave resonance absorption of the fastener in Article 0. As a result, a top plate designed to maximize vibration isolation may be less effective at controlling rail vibration and noise than a relatively weak top plate that resonates at frequencies between 500 and 1000Hz, the important frequency range of tangent track rolling noise.

The measured vertical transfer dynamic stiffness of an early design bonded fastener is plotted in **Figure 9.3.3** for various static loads. The transfer stiffness is defined as the transmitted force relative to the deflection of the rail web (or centroid of the rail). The measured static stiffness of the fastener was about 300,000 lb/in [53 KN/mm] at static loads between 2,000 and 5,000 lb [9 KN and 22 KN]. The nominal dynamic stiffness was about 500,000 to 600,000 lb/in [87.5 KN/mm to 105 KN/mm] between 200 and 500 Hz. The maxima of these transfer stiffness curves at 700 to 750 Hz are due to resonance of the top plate that increases the transfer stiffness of the fastener by as much as a factor of 3 or 4 to about 1,400,000 lb/in [245 KN/mm] at the resonance frequency. This resonance amplifies transmitted forces and structure-borne noise and vibration. The corresponding input stiffness to the top plate was not measured and should not be confused with the transfer stiffness.

This mode of vibration was investigated with a finite element model. The nominal elastomer stiffness of 267,000 lb/in [47 KN/mm] was assumed to be distributed uniformly under the top plate, which measures 8 inches [203 mm] longitudinal with the rail by 18 inches [457 mm]

transverse to the rail by 0.5 inch [12.5 mm] thick. The rail section is 115RE, modeled as a 30-inch [762-mm] long section, free at either end. For this model, the computed resonance frequency of the rail and top plate in vertical translation is 147 Hz, but bending of the top plate reduces the stiffness and brings the computed resonance frequency down to 140 Hz. (The length of 30 inches [750 mm] was chosen to represent the tributary portion of the rail for a given fastener. This approach is not an accurate representation because the rest of the rail and fasteners that would affect the resonance of the rail in bending are not included.)

Figure 9.3.4 illustrates the bending mode associated with top plate bending resonance. In this example, the top plate is bending symmetrically about the vertical rail center plane, with little motion by the rail. The fundamental mode frequency is 561 Hz, at which frequency amplification of transmitted forces would occur. The calculated resonance frequency may be compared with the peak in the transfer stiffness test data shown in Figure 9.3.3**.**

The top plate bending stiffness may be increased by both reducing the transverse dimension of the top plate and increasing its thickness. **Figure 9.3.5** illustrates the top plate bending resonance with a top plate of 0.75 inch [19 mm] thickness and 12 inches [304 mm] transverse dimension and 8 inches [200 mm] longitudinal dimension. The rail is again 30 inches [750 mm] of 115 RE rail, and the nominal stiffness of the fastener is again 267,000 lb/in [47 KN/mm]. This mode may be compared with the mode shape for the 0.5-inch [12.5-mm] thick top plate shown in Figure 9.3.4. This mode of vibration involves symmetric bending of the fastener top plate about the rail vertical center plane. The modal resonance frequency is now 1,433 Hz, which may be compared with the frequency of 561 Hz obtained for the symmetric mode shown in Figure 9.3.4. In this case, a substantial portion of the bending energy is in the rail as well as the top plate, as the rail is bending over the fastener. A better view of this mode is provided in **Figure 9.3.6.**

The top plate resonance frequency is controlled by the bending stiffness of the plate, the transverse bending stiffness of the rail base, and the stiffness of the elastomer. The resonance frequencies of the above examples would be less with softer elastomer. The difference with the thick, stubby top plate would less, as the stiffness of the top plate is much higher than that of the elastomer to begin with.

A high-resonance-frequency top plate requires that the overall thickness of the fastener be large enough to allow for a thick top plate. A minimum of 2 inches [50 mm] should be made available between the rail base and invert for high-compliance fasteners. The elastomer strain can be kept to a practical minimum under static load with a 2-inch [50-mm] thick fastener. Increasing the thickness of the top plate should increase its strength and reliability, with less working of the rail clip due to top plate flexure under static load and strain vibration. Thickening the top plate will increase the cost of top plate castings, although the increase should be relatively small in comparison to the overall cost of the fastener.

Fasteners designed for vibration isolation should have stiff top plates that avoid increasing the transfer stiffness of the fastener and amplifying vibration forces in the frequency of structure-borne noise and ground-borne noise. Dissipation in the soil will reduce high-frequency ground-borne noise, so ground–borne noise is less of an issue than structure-radiated noise. Resilient fasteners with stiff top plates will minimize structure-radiated noise from aerial structures with

steel box or I-Beam girders or steel elevated structures. However, this has to be balanced against the loss of rail vibration absorption at frequencies in the range of 500 to 1,000 Hz.

Some fasteners support the rail with elastomer shoulders, such that no fastener metal is in contact with the rail. This type of fastener can provide very low stiffness and support the rail with minimal clearance between the rail and invert. The elastomer isolates the rail from the mass of the fastener base plate and invert. At high frequencies, the effect is as though the rail is freely suspended in space. The input mechanical impedance of such a rail consists in equal parts of reactive and resistive impedance,[62] making the rail behave as though it were well damped.

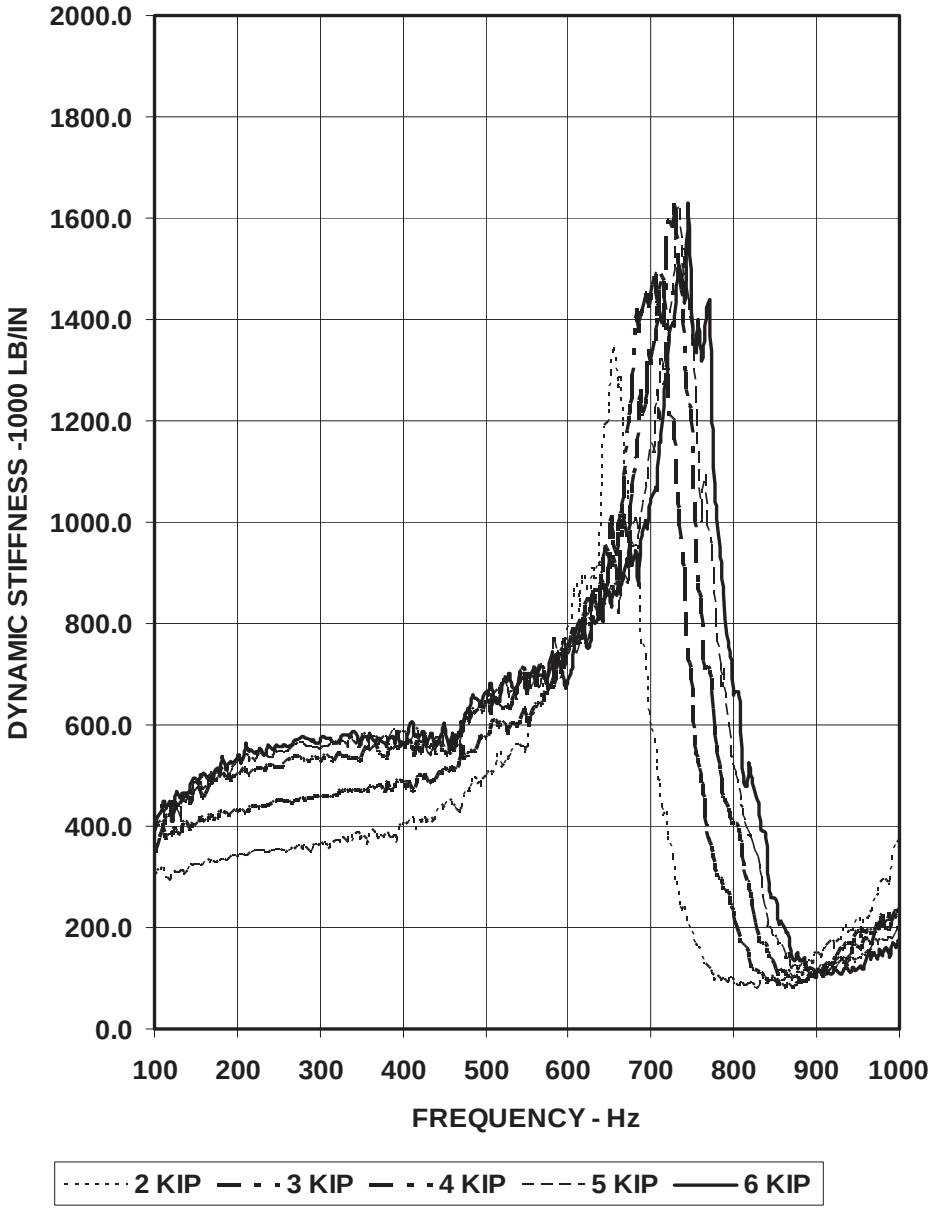

Figure 9.3.3 Dynamic transfer stiffness of bonded direct fixation fastener of nominal static stiffness of 400,000 lb/in [70 KN/mm]

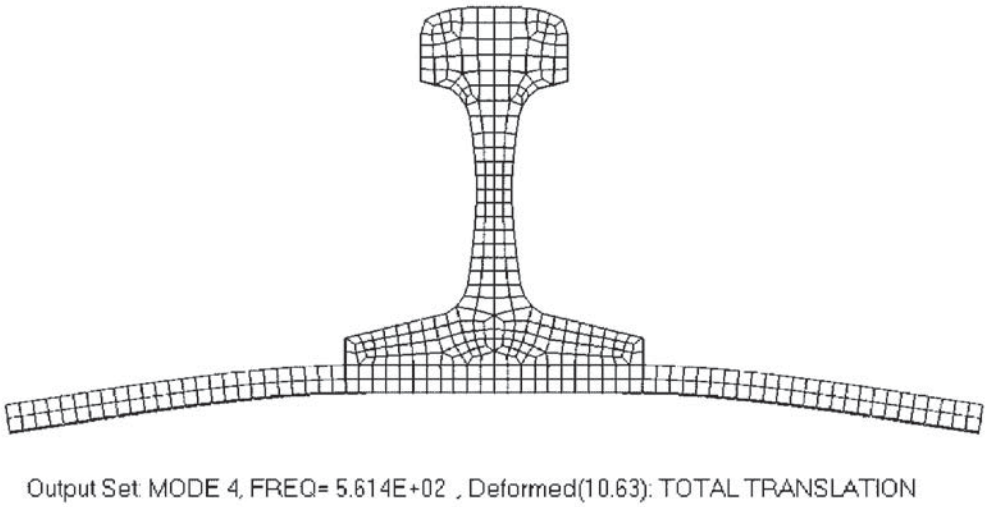

Figure 9.3.4 Mode shape of an idealized fastener consisting of a 0.5-inch [12.5-mm] thick by 18-inch [304-mm] long by 8-inch [203-mm] wide top plate supporting a 30-inch [750-mm] long section of 115 RE rail

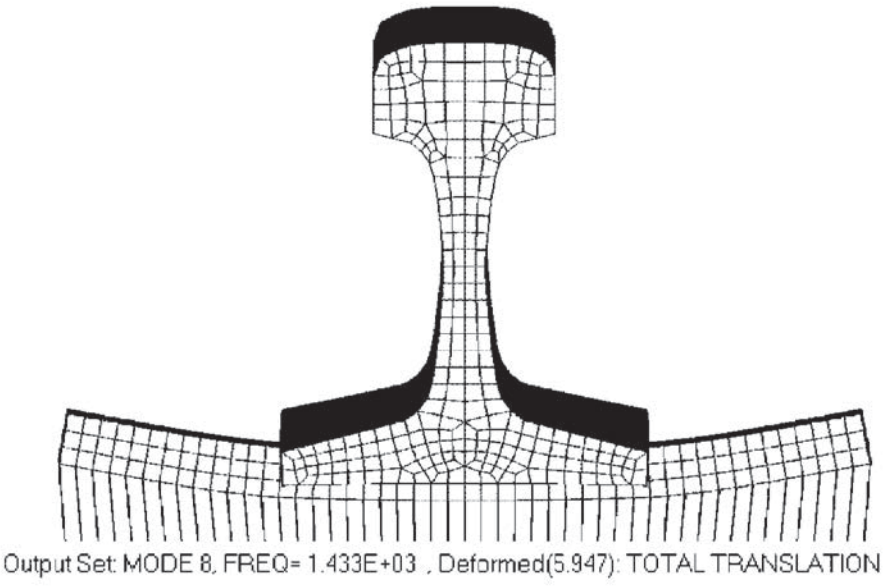

Figure 9.3.5 Mode shape of an idealized fastener consisting of a 0.75-inch [19-mm] thick by 12-inch [304-mm] long by 8-inch [203-mm] wide top plate supporting a 30-inch [750-mm] long section of 115 RE rail

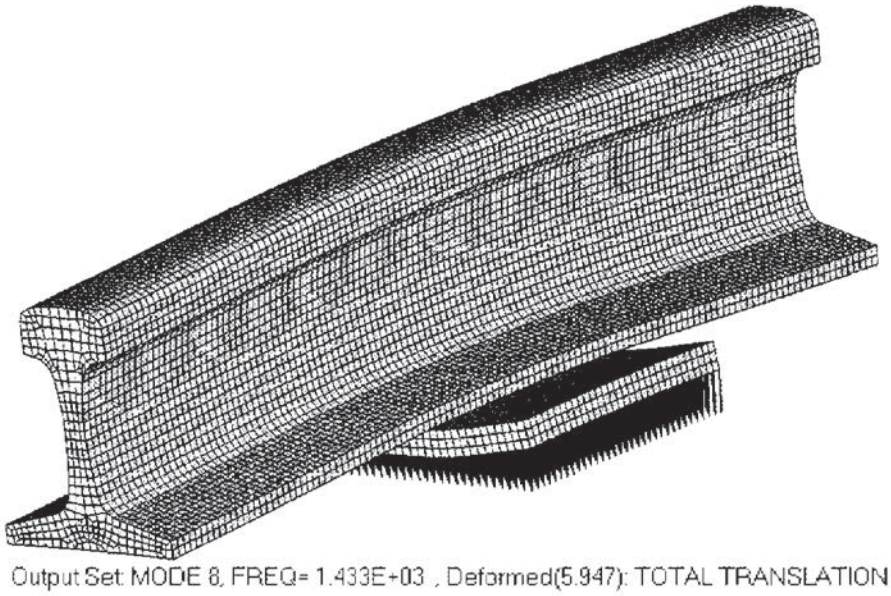

Figure 9.3.6 Rotated view of Figure 9.3.5

9.3.3.6 Rail Grinding

Rail grinding to eliminate checks, spalls, and undulation of the rail head reduces ground-borne noise and vibration, provided that the vehicle wheels are well maintained. This applies especially to corrugated rail. Rail grinding to reduce ground vibration at low frequencies must remove long wavelength roughness and corrugation, which may require special grinders with long grinding bars or special controls. Rail grinding will have the greatest benefit at frequencies above 15 to 30Hz for normally worn rail, including rail with short pitch corrugation. The benefit at low frequencies is relatively modest, due to the wavelengths involved. However, removing checks and spalls would be expected to reduce vibration across a broad frequency range, including low frequencies.

9.3.3.7 Rail Undulation

Although often overlooked or not considered during track design, rail straightness is fundamentally important in controlling ground vibration below 10 Hz in critically sensitive areas such as university campuses, semiconductor manufacturing plants, and research institutes. Roller-straightened rails have produced ground vibration spectral components that can be related to the straightener roller diameter. Substantial vibration below 10 Hz was generated by unit trains after replacing "gag-press" straightened rail with roller-straightened rail with excessive vertical undulation at Kamloops.[63] The result was substantial community reaction and litigation. Narrowband analyses of the wayside ground vibration data identified a linear relation between frequency peaks and train speed that related directly to the roller diameters of the straightening machine and rolls. Subsequent field measurements of rail profile with a laser collimator confirmed the vibration data. The roller-straightened rail was replaced with new rail that was also roller straightened, but to CORUS (formerly British Steel) specifications. Repeat measurements

indicated a substantial reduction of ground vibration, even though the effects of the roller straightener pitch diameter were still identifiable in the wayside ground vibration spectra.

This experience suggests using "super-straight" rail for sensitive areas where a low-frequency vibration impact is predicted. Examples include alignments in very close proximity to sensitive receivers of all types in areas with very soft soil, alignments adjacent to or through university campuses where engineering and scientific research occurs, and alignments in proximity to sensitive manufacturing facilities, such as semiconductor manufacturing. As manufacturing technology and fundamental research advances, this problem will become more important.

Controlling low-frequency vibration due to rail undulation by controlling rail straightness should be far less costly than the installation of a floating slab track structure. Soft fasteners would provide no positive benefit and may even exacerbate low-frequency vibration of this type. Corrective rail grinding is incapable of removing rail height undulation over long wavelengths of 6 feet or more. Rail straightness specifications may likely be included in procurement specifications for rail destined for high-speed rail systems, as excessively undulating rail would require additional maintenance of track and perhaps the subgrade. Thus, straight rail will likely become more available with time. Suggested limits on peak-to-valley undulation are listed in **Table 9.3.2.** These may apply to both horizontal and vertical deviations. The rail manufacturer should be consulted with respect to manufacturing capability and quality assurance.

Table 9.3.2 Rail undulation limits

Length	80 inches [2 m]	120 inches [3 m]
Limit	0.008 inch [0.2 mm]	0.12 inch [0.3 mm]

9.3.3.8 Vehicle Primary Suspension Design

Vehicle primary suspension design is not part of track design, but has a direct bearing on wayside ground vibration amplitudes. Selection of trackwork vibration isolation provisions should ideally be based on the type of vehicle involved. In general, vehicles with soft primary suspensions produce lower levels of ground-borne noise and vibration than vehicles with stiff suspensions. Differences in suspension characteristics may be sufficient to eliminate the need for floating slab isolation at otherwise critically sensitive locations. Introduction of vehicles with stiff primary suspensions relative to existing vehicles with soft suspensions may introduce vibrations in the 10- to 25-Hz frequency region.

The selection of chevron-type suspension systems in lieu of stiff rubber journal bushing suspension systems may provide sufficient vibration reduction to reduce the need for other vibration isolation provisions in the frequency range of about 16 to 31.5 Hz. Most modern light rail transit vehicles in the United States incorporate chevron primary suspension systems with low vertical stiffness, thus reducing the demand on vibration isolation elements in the track. If the vehicles have stiff primary suspension systems, such as rubber journal bearing springs, particular attention should be paid to low-frequency vibration control in track at the primary suspension resonance frequency.

9.3.3.9 Resilient Wheels

Resilient wheels might provide some degree of vibration isolation above 80 Hz relative to solid steel wheels, depending on elastomer stiffness, by reduction of the unsprung mass. Resilient wheels actually modify the P2 resonance, or track resonance, by introduction of another mass-spring element between the rail and axle or wheel center. The resilient wheel will introduce an additional resonance of the axle mass and wheel center on the resilient wheel springs, and this resonance may amplify ground vibration at the resonance frequency. Details on this have not been investigated, but the resonance frequency was calculated to be about 50 Hz for the axle and wheel centers with Bochum 54 wheels. Strong spectral peaks at the 50 and 63 Hz ⅓-octave bands have been observed in the wayside ground vibration spectrum for light rail vehicles at some systems with resilient wheels and no spectral peaks at others using the same wheel. Rail corrugation would be the likely cause, and other factors are likely important as well. More research is required to further define the cause of this type of corrugation and determine which, if any, track design parameters may influence its generation. Resilient wheels are widely used on light rail systems because they control wheel squeal at curves and reduce truck vibration.

9.3.3.10 Subgrade Treatment

The vibration amplitude response of soil is roughly inversely proportional to the stiffness of the soil. Therefore, stiff soils tend to vibrate less than soft soils. Grouting of soils or soil stabilization with lime for organic soils or cement for sandy materials is attractive where very soft soils are encountered. However, grouting may increase the efficiency of high-frequency vibration propagation between track and building structures, so care must be exercised in treatment design. Grouting of soils has been conducted at high-speed lines in Europe for vibration control, and rail-car-mounted grouting systems may be available.

9.3.3.11 Special Trackwork

Turnouts and crossovers are sources of vibration. As the wheels traverse the frogs and joints, impact forces are produced that cause vibration. Grinding the frog to maintain contact with a properly profiled wheel can minimize impact forces at frogs. Spring frogs and movable point frogs are designed to maintain a continuous running surface. Spring frogs are practical for low-speed turnouts, while movable point frogs are more suited to high-speed turnouts. Refer to Chapter 6 for additional discussion on frog types. Also, refer to Article 9.2.2 for discussion of impact noise generation and details affecting frog designs.

9.3.3.12 Distance

The track should be located as far from sensitive structures as practicable within the limits of the right-of-way. Where wide rights-of-way exist, some latitude in locating the track may exist. A shift of as little as 10 ft [3 m] away from a sensitive structure may produce a beneficial reduction of vibration for receivers bordering the right-of-way. Sensitive receivers located within 50 ft [15 m] of the track centerline are particularly in danger of being impacted by ground vibration from transit operations. However, ground vibration and ground-borne noise impacts have occurred at sensitive receivers at significantly greater distances.

9.3.3.13 Trenching

Open trenches have been considered for vibration reduction, but are of limited effectiveness below 30 Hz for a depth of 20 ft [7 m] and even less for shallower trenches. At higher

frequencies, the vibration reduction of a trench filled with Styrofoam may be as little as 3 to 6 dB. Concrete barriers embedded in the soil have also been considered. While they may interrupt surface wave propagation, their mass must be substantial to provide sufficient vibration reduction. Detailed finite element modeling is necessary in this case to predict performance. A number of theoretical studies of vibration barrier insertion losses have been conducted and continue to be studied.[64] [65] [66] Practical design guidelines are provided by Richart et al.[67] In particular, the trench depth must be similar to the Rayleigh surface wave length to obtain an appreciable vibration reduction on the order of 12 dB (75% reduction). For a Rayleigh wave propagation velocity of 400 ft/sec [122 m/s] and excitation frequency of about 8 Hz, the primary suspension resonance frequency of a typical chevron suspension system, the wavelength would be 50 ft [15 m]. Thus, the trench would have to be on the order of 50 ft [15 m] to be effective. On the other hand, the Rayleigh wavelength at the track resonance frequency of 50 Hz, responsible for ground-borne noise, would be about 8 ft [2.4 m], and the depth of the trench would also have to be on the order of 8 ft [2.4 m]. Thus, trenches might be most practical for controlling ground-borne noise as opposed to low-frequency perceptible vibration. An example of vibration isolation provided by trenching is presented in **Figure 9.3.7**.

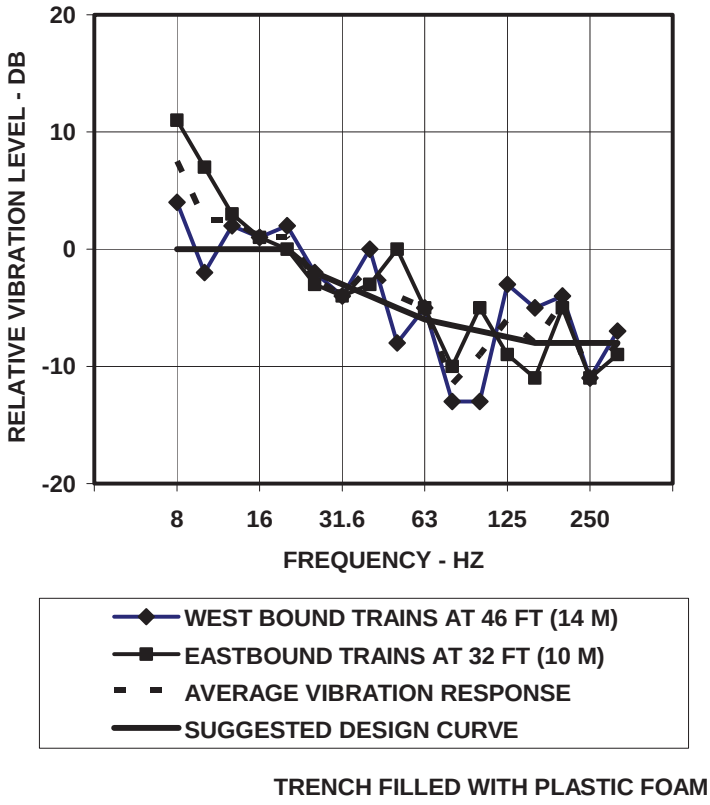

Figure 9.3.7 Measured ground vibration insertion gain of styrofoam-filled trench at Toronto Transit Commission

These data were obtained by the TTC for evaluation of a 14-ft [4.3-m] deep trench containing a 4-inch [100-mm] thick layer of Styrofoam and back-filled with soil.[68] The test train was a light rail vehicle on surface ballast and tie track. Test distances were 22 to 32 ft [6.7 to 9.8 m] from the

near track. The results are averages for eastbound and westbound operation. A suggested design curve is shown. However, finite element modeling should be conducted as part of any trench design since results depend on soil stiffness and distances of both track and receiver from the trench. For example, the trench would be ineffective for receivers at large distances from the track.

9.3.3.14 Pile-Supported Track

Piling used to reinforce a track support system can be effective in reducing ground vibration over a broad range of frequencies. An example would be a concrete slab track supported by piles or ballasted track on a concrete trough supported by piles. Performance improvement is likely to be substantial if the piles can be extended to rock layers within about 65 ft [20 m] of the ground surface or foundation. Standing wave resonances may occur in long piles, so there is a limit on the effectiveness of piles in controlling audible ground-borne noise. Unfortunately, piles may interfere with utilities. Piling may be attractive for civil reasons, and the added benefits of vibration control can be realized with appropriate attention to design.

9.4 WHEEL/RAIL PROFILES AND CONTACT STIFFNESS AND STRESS

Wheel and rail contact geometry affects the traction, or pressure, distribution in the contact area, stress distributions in the rail head, and the contact stiffness. The contact stiffness affects the dynamic interaction and vibration of the wheel and rail in response to surface roughness. Maintaining a low contact stiffness helps to maintain low dynamic contact forces and thus low wheel and rail vibration. Wayside noise and ground vibration are directly proportional to wheel and rail vibration. Thus, low contact stiffness is important for controlling noise and vibration.

Surface defects and plastic deformation are related to stress distributions in the rail head and contribute to roughness of the rail running surface. Controlling stresses in the rail is thus of interest from a maintenance as well as noise control point of view. However, as discussed below, minimizing the magnitude of the stress is not necessarily consistent with minimizing the contact stiffness.

Rail corrugation is a complex regenerative process that may affect both soft and hard rails. Rail corrugation may be due to abrasion or excessive stresses, depending on the corrugation process. The corrugation process is not clearly understood in many cases, but wheel and rail vibration necessarily influence corrugation and wear. The specifics of rail corrugation are discussed in Article 9.2, specifically Article 9.2.1.1.3.

This discussion uses Hertzian contact theory to explore the relationship between contact geometry, contact stiffness, and stress at tangent track where short-pitch corrugation generally occurs. The tire radius and rail head curvature can be well controlled on tangent track by grinding and wheel truing, so Hertzian contact theory can be applied in a straightforward way.

Tangential tractions due to lateral creep, curving, braking, and acceleration are not discussed here due to their complexity; nevertheless they are important as well.

9.4.1 Contact Dimensions

The wheel/rail Hertzian contact tractions are functions of the wheel's rolling radius (or diameter), transverse tread profile, the rail head crown radius, the modulus of elasticity of steel, and the wheel load. The wheel/rail contact geometry is elliptical in shape with major and minor axes aligned with the rail or transverse to the rail, depending on the radii of curvature of the contacting surfaces. The contact dimensions are calculated with Hertzian contact theory, excellent discussions of which are provided by Timoshenko and Goodier[69] and by Johnson.[70]

Assuming a constant radius of curvature of the wheel tread profile and rail head running surface, the contact geometry and equal-stress contours in the contact zone will assume concentric ellipses. Representative major and minor axes of the contact area for a 28-inch [711-mm] diameter wheel with linear tread profile for various rail head radii are plotted in **Figure 9.4.1**. The principal axes of the contact ellipse are equal when the rail head radius equals the tire radius with a straight tread profile. In this case, the contact area is a circle, and the equal-stress and equal surface traction contours would also be circles. As the head radius is reduced, the dimension of the contact parallel with the rail increases, and the dimension transverse to the rail decreases. For a 9-inch [229 mm] head radius, the longitudinal and transverse major axes of the edge of the contact ellipse are 0.4 and 0.3 inch [10.2 and 7.6 mm], respectively.

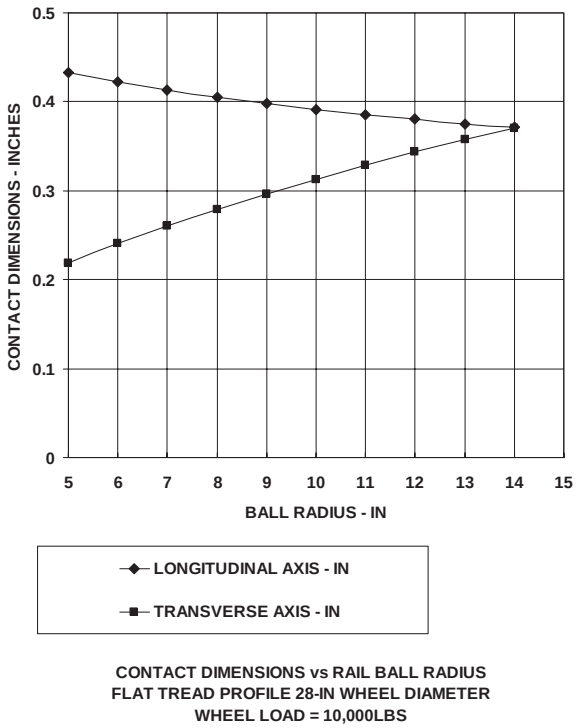

Figure 9.4.1 Contact geometry vs. rail head radius for a 28-inch [711-mm] diameter wheel with linear tread profile

Figure 9.4.2 illustrates the contact area as a function of head radius for three wheel tread profiles: linear, concave with a 28-inch [711-mm] radius, and concave with a 14-inch [356-mm]

radius. The contact area is relatively insensitive to head radius for wheel treads with linear profiles and concave profiles of 28 inches [711 mm]. However, with a concave profile of 14 inches [356 mm], the contact area increases rapidly with head radius above 10 inches [250 mm]. The contact area approaches 0.3 inch^2 [194 mm^2] as the head radius approaches 13.9 inches [153 mm], a condition of almost fully conformal contact. The contact ellipse dimensions in this case are 2.71 inches [68.8 mm] transverse and 0.15 inch [3.8 mm] longitudinal. At 14-inches [356-mm] tread radius, the contact area is undefined, as the contact geometry is a line contact across the rail head. Interestingly, the longitudinal and transverse axes are approximately equal at 0.37 inch [9.4 mm] with a 7-inch [178-mm] rail head radius. The contact area in this case is about 0.1 inches^2 [64.5 mm^2].

9.4.2 Stresses

Two stresses are produced by a vertical load on the rail surface. One is the principal stress balancing the normal traction at the contact and the other is the maximum shear stress, which is at some distance beneath the surface of the rail.

9.4.2.1 Normal Stress

A positive contact pressure, or normal traction, is balanced by a negative stress in the rail at the surface. This is the result of a sign convention, where a positive principal stress is due to stretching of the material, and a negative principal stress is due to compression. The stresses are discussed here in terms of pressure.

The maximum contact pressure occurs at the center of the contact and is theoretically 50% higher than the average pressure over the contact zone. The maximum contact pressure and average pressure versus rail head crown radius is plotted in **Figure 9.4.3** for a 28-inch [711-mm] diameter wheel with linear (flat) tread profile. The variation of maximum pressure with head radius for linear and concave tread profiles is plotted in **Figure 9.4.4**. The maximum contact pressure decreases monotonously with increasing head radius. The stresses are highest with the linear (flat) wheel tread profile, ranging from 200,000 psi [1,424 MPa] with a 5-inch [127-mm] crown radius, down to about 140,000 psi [997 Mpa] with a 14-inch [356-mm] crown radius. The lowest contact pressure maximum occurs with a 14-inch [356-mm] radius concave tread profile. In this case, the contact pressure declines rapidly to 50,000 psi as the head radius approaches 14 inches [356 mm], the fully conformal contact condition.

The above pressures are comparable with endurance limits for rail steel in good condition and above the limit for rail steel in moderately corroded condition, which limit may be as low as 60,000 psi [127 Mpa]. Timoshenko calculates a contact pressure of 63,000 psi [449 Mpa] for a 1,000-lb [4,448-N] wheel load, where the wheel radius is 15.8 inches [400 mm] and the rail head radius is 12 inches [305 mm]. (Timoshenko, as it turns out, published much on rail stresses.) So, even at a 1,000-lb [4,448-N] load, the contact pressures of a typical railroad rail/wheel combination are at the endurance limit. The maximum contact pressure increases as the cube root of the load, so the maximum contact pressure at a 10,000-lb [44,480-N] load would be 136,000 psi [968 Mpa]. Reducing the wheel diameter and head radius simply increases the maximum contact pressure.

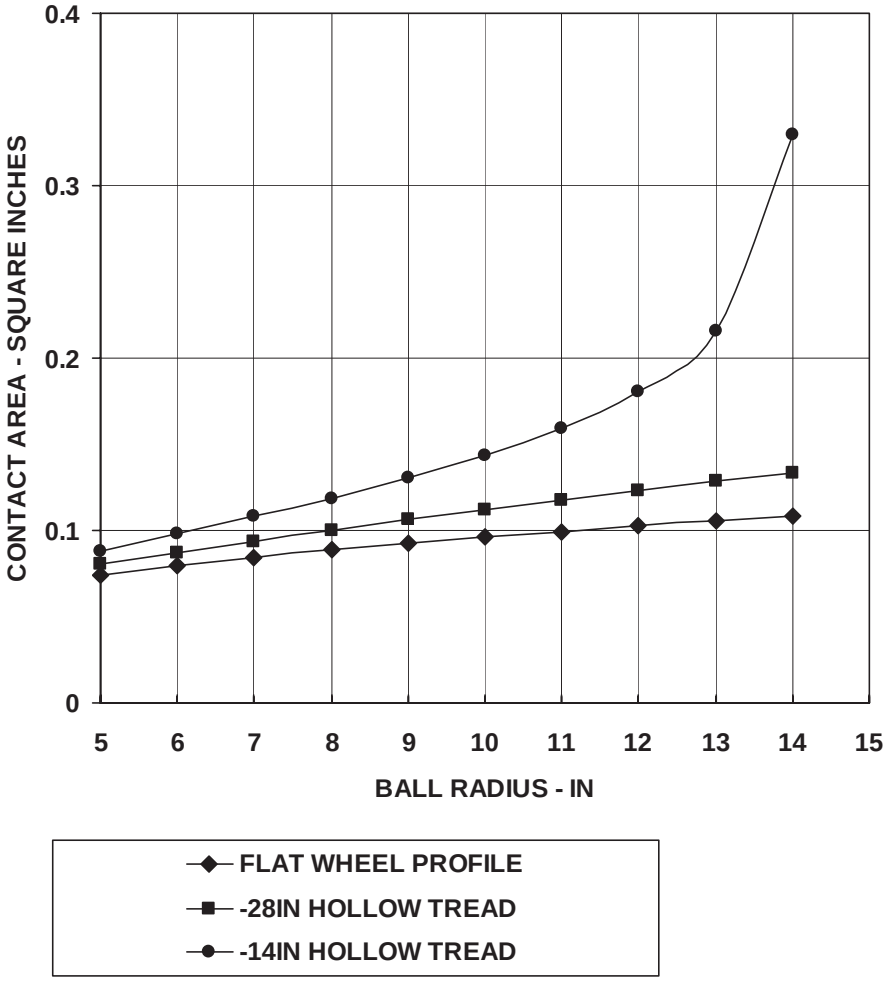

Figure 9.4.2 Contact area vs. tread profile

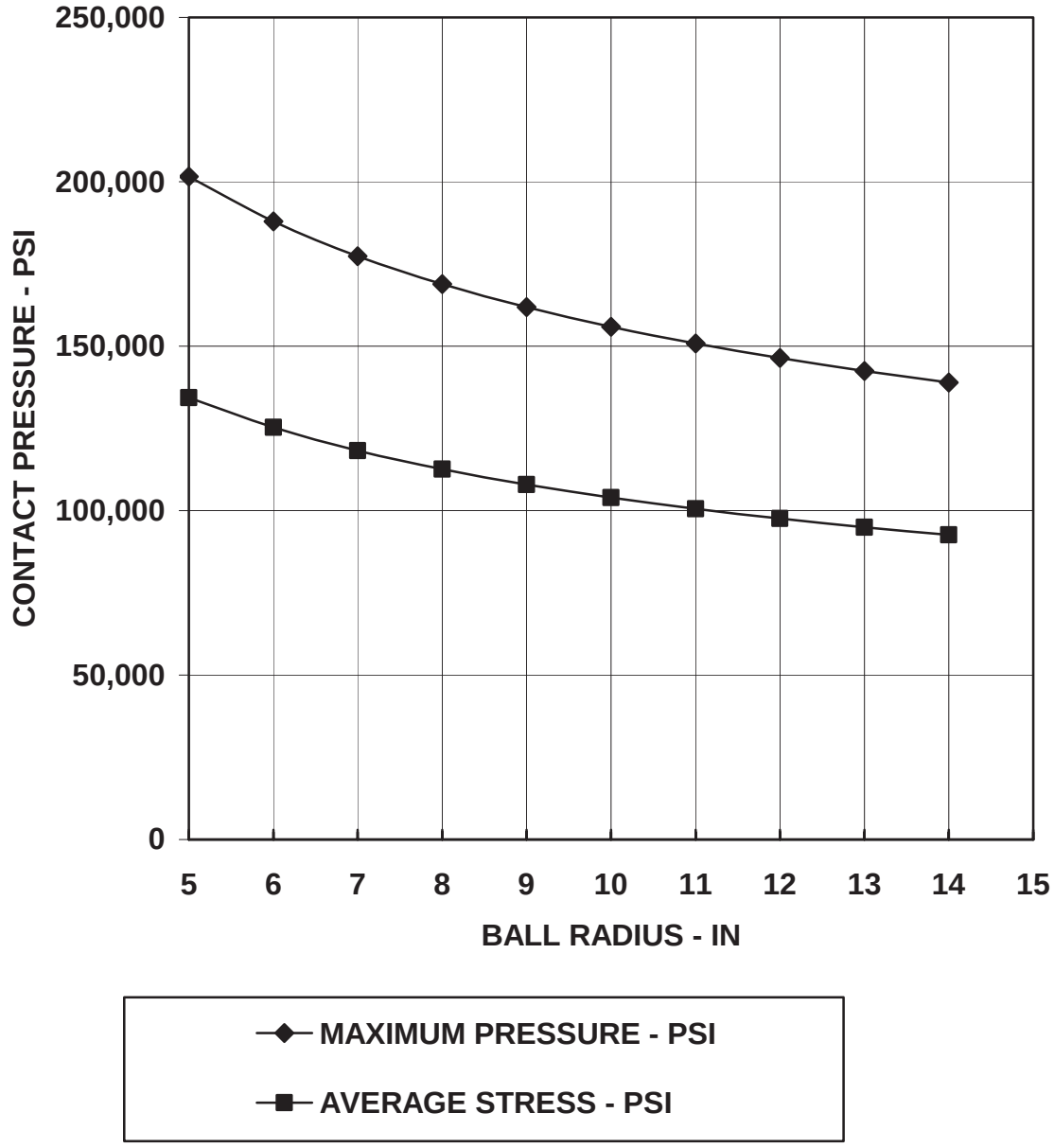

Figure 9.4.3 Maximum contact pressure vs. head radius for 28-inch [711-mm] diameter linear tread radii

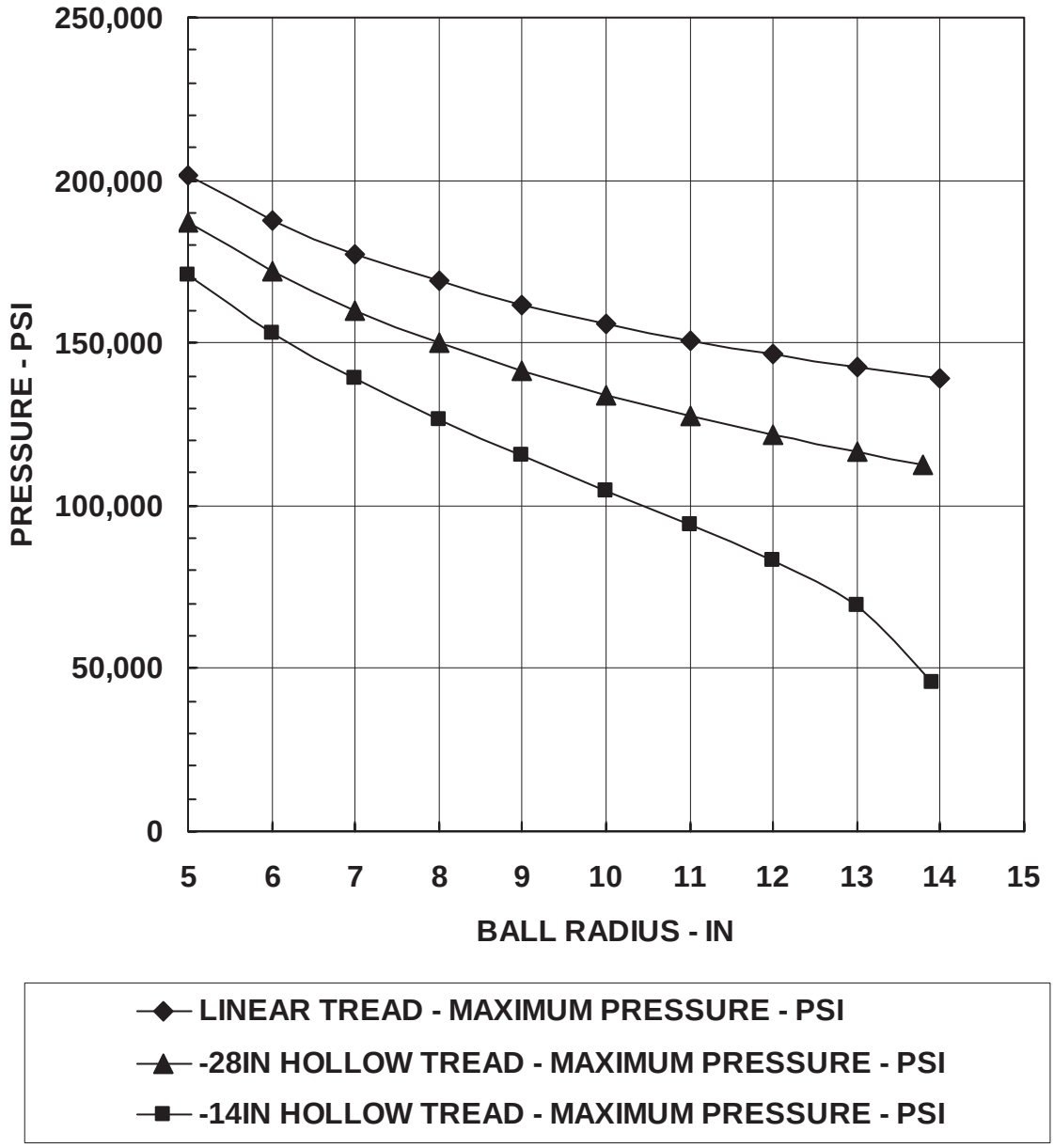

Figure 9.4.4 Maximum contact pressure vs. rail head radius for various concave tread radii

The depth of the negative stress (compressive stress) distribution increases as the contact geometry approaches a line contact condition, which may occur with concave tread profiles that match the rail head profile or in curves where the throat and gauge corner radii match.

9.4.2.2 Shear Stress

The shear stress is very important with respect to metal fatigue. The point of maximum shear occurs below the center of the contact area for nominally circular contact areas. For this reason, running surface fatigue and rail failures often begin at some point beneath the surface.

The depth of maximum shear stress is estimated by Kerr[71] to be 0.6 times the radius of the contact area, assuming a uniform distribution of contact pressure (which is not an entirely realistic assumption, but facilitates the discussion). Timoshenko gives a depth of 0.47 times the contact radius for circular contacts.

Assuming a contact mean dimension on the order of 0.37-inch [9.4-mm]—as indicated in Figure 9.4.1 for a 14-inch [356-mm] rail head radius and a 28-inch [711-mm] diameter wheel with linear tread profile (this corresponds to a circular contact area)—the depth of maximum shear stress, according to Timoshenko, would occur at about 0.174 inch [4.4 mm] below the rail surface.

The shear stress distribution is more complicated for high aspect ratio elliptical contact areas. **Figure 9.4.5** illustrates the variation of the depth below the contact surface at which the maximum shear stress occurs and the maximum shear stress variation as a function of the ratio of the minor and major axes of the contact ellipse. For a line contact, the depth to the point of maximum shear would be about 0.8 times one half of the minor axis length, although the minor axis is ill-defined in this case. This is an extreme case, but represents the condition of fully conformal contact. The ratio decreases to about 0.485 for circular contacts, where the major and minor axes are equal. Assuming a contact area radius of 0.37 inch, the depth below the surface would be 0.18 inch [4.6 mm], in good agreement with the result given by Timoshenko.

The ratio of maximum shear stress to maximum contact pressure is also plotted in Figure 9.4.5 as a function of the contact area aspect ratio. The ratio of the maximum shear stress to the maximum contact pressure at the surface remains relatively constant at about 0.31 to 0.32 as a function of the ratio of minor to major axes. Thus, for a contact pressure of nominally 150,000 psi [1067 MPa], the shear stress below the surface would be about 50,000 psi [356 MPa]. To the extent that increasing the aspect ratio contact would tend to reduce contact pressure, the shear stress would also be reduced, regardless of the aspect ratio.

The maximum shear stress versus rail head crown radius is plotted in **Figure 9.4.6** for three wheel tread profiles: linear, concave with 28-inch [711-mm] radius, and concave with 14-inch [356-mm] radius. These were obtained from the maximum contact pressures plotted in Figure 9.4.4 and the ratios plotted in Figure 9.4.5. The wheel diameter is again assumed to be 28 inches [711 mm]. The linear tread profile produces the highest shear stress, ranging from 65,000 psi [463 MPa] for a head radius of 5 inches [127 mm], down to about 43,000 psi [306 MPa] for a head radius of 14 inches [356 mm]. These stresses are reduced to about 60,000 psi [427 MPa] and 37,000 psi [263 MPa], respectively, for a concave tread profile with 28-inch [711-mm] radius. The shear stresses with a concave tread of 14 inches [356 mm] in radius are

lower still, ranging from 55,000 psi [391 MPa] with a 5-inch [127-mm] head radius down to less than 30,000 psi [214 MPa] with a 12-inch [305-mm] head radius.

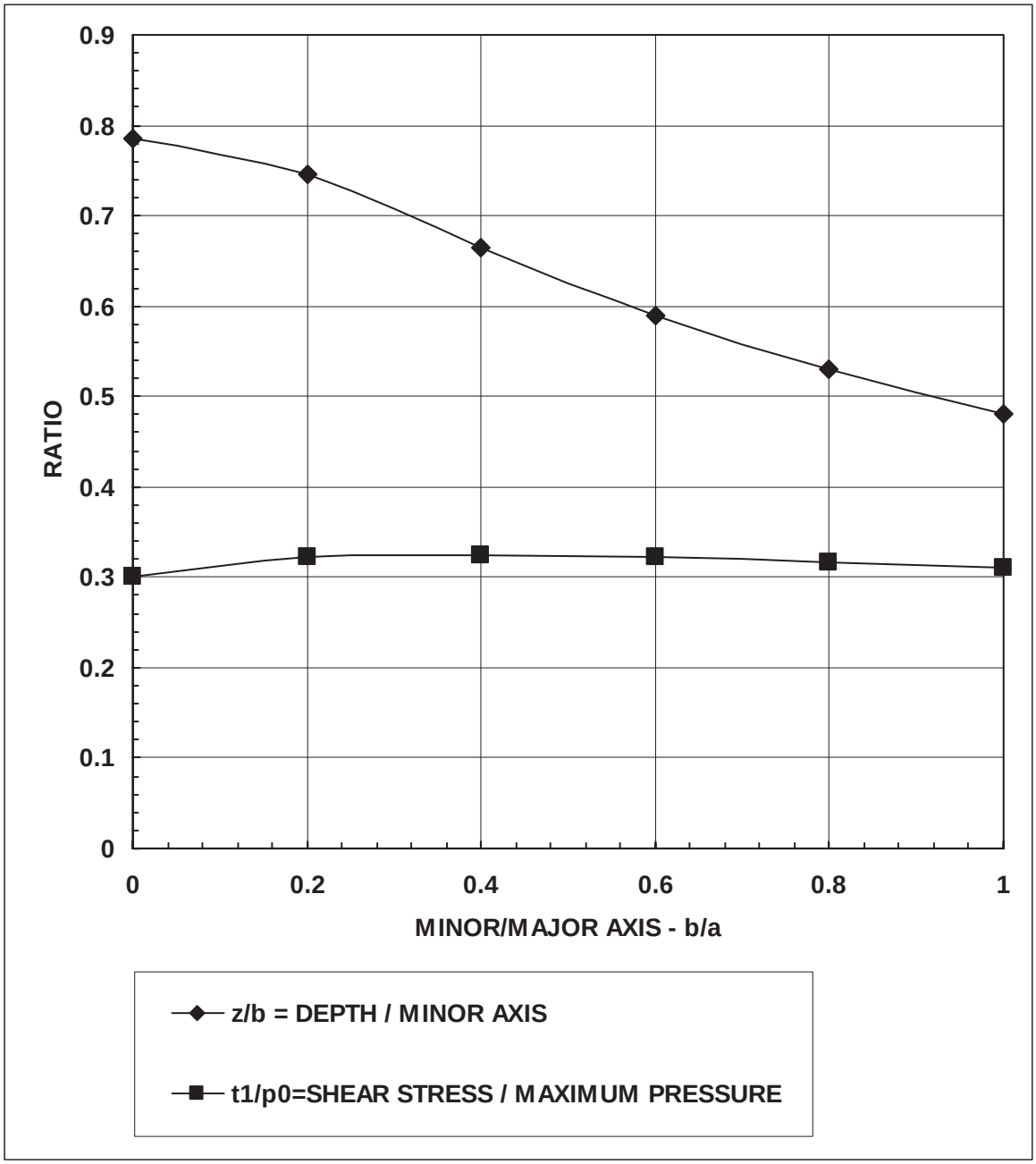

Figure 9.4.5 Depth of maximum shear and maximum shear vs. contact ellipse aspect ratio (Johnson, 1992)

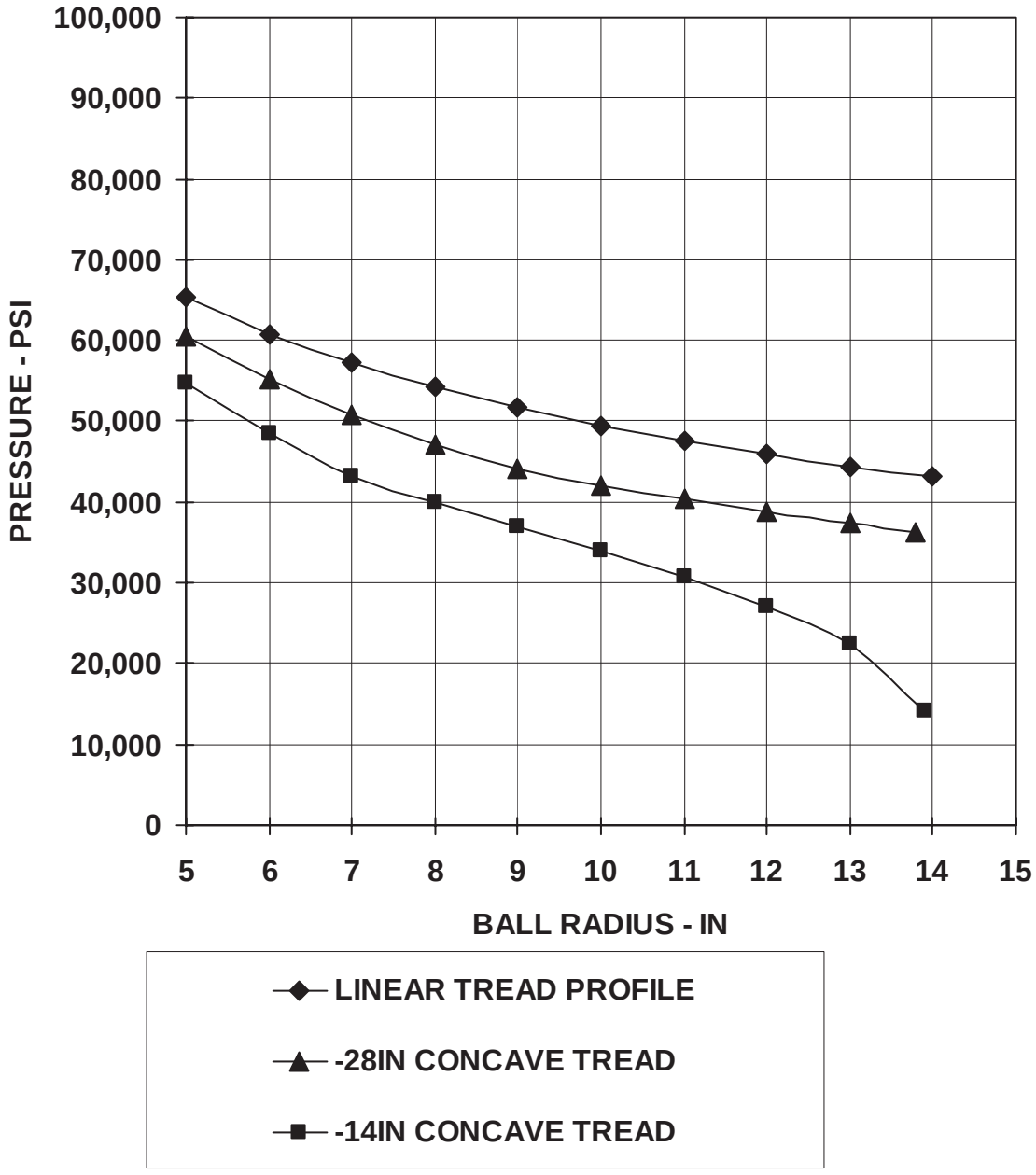

Figure 9.4.6 Maximum shear stress vs. head radius for three tread profiles

As the head radius increases further, the shear stress drops rapidly, becoming poorly defined at the fully conformal contact conditions with a 14-inch [356-mm] head radius.

The fatigue limit for shear stress is roughly about 50% of the fatigue limit in tension. If a rail steel has a fatigue limit of about 100,000 psi [825 MPa], the fatigue limit would be about 50,000 psi [345 MPa]. The shear stress would be at a minimum of 43,000 psi [296 MPa] for a fully rounded contact area achieved with a 14-inch [356-mm] rail crown radius and 28-inch [711-mm] wheel with linear tread, less than the shear fatigue limit. Introduction of a radius in the tread profile, producing some degree of conformal contact, reduces this further.

While conformal contact might appear to be desirable to reduce contact pressures and shear stresses, increasing conformal contact increases contact stiffness, which increases *dynamic* forces that are superimposed on the static load, thus increasing contact *dynamic* pressures and *dynamic* shear stress. This is discussed further below.

9.4.3 Contact Stiffness

The dynamic contact stiffness at a given load is the increment in contact force produced by an increment in relative deflection of the wheel tread and rail centers of gravity, or neutral axes. The dynamic contact stiffness is equal to 1.5 times the static load divided by the static relative deflection of the wheel and rail. The static stiffness divided by the static deflection is a function of load as well. Thus, the dynamic Hertzian contact stiffness is non-linear. The deflection is proportional to the two-thirds root of the load. Thus, the dynamic contact stiffness is proportional to the cube root of the load.

The contact stiffness as a function of rail head radius is plotted in **Figure 9.4.7** for the three tread profiles considered above. The contact stiffness remains between about 5,000,000 lb/in [875 KN/mm] and 7,000,000 lb/in [1,226 KN/m] for rail head radii between 5 inches [127 mm] and 14 inches [356 mm] for the linear and 28 inches [711 mm] radius concave tread profile and wheel radius of 28 inches [711 mm]. For the 14-inch radius tread profile, the contact stiffness increases rapidly with rail crown radii greater than about 13 inches [330 mm], becoming very large as the radius approaches 14 inches [711 mm]. That is, as the rail head and wheel tread contact approach the condition of fully conformal contact, the contact stiffness increases very rapidly. As the contact stiffness increases, high-frequency vibration of the wheel and rail and associated dynamic stresses increase. Avoiding fully conformal contact would appear to be desirable from a track and wheel design point of view.

The contact mechanical impedance is obtained by dividing the contact stiffness by the product of two, pi, and the frequency in Hz ($2\pi f$). At corrugation frequencies on the order of 1,000 Hz, the contact mechanical impedance of a contact stiffness of 6,000,000 lb/in [1050 KN/mm] would be about 954 lb-sec/in [0.167 KN-sec/mm], roughly comparable with the mechanical impedances of the wheel and rail as shown in Figure 9.2.7, Figure 9.2.9, and Figure 9.2.10. Rail corrugation commonly occurs at a frequency of about 800 Hz. The mechanical impedance of a rigid wheel weighing 500 lb [227 Kg] at 800Hz would be 6,511 lb-sec/in [1.1 KN-sec/mm]. The contact mechanical impedance at 800 Hz would be about 1,200 lb-sec/in [0.21 KN-sec/mm], much smaller than that of the wheel. Resonances of the wheel would complicate this analysis considerably.

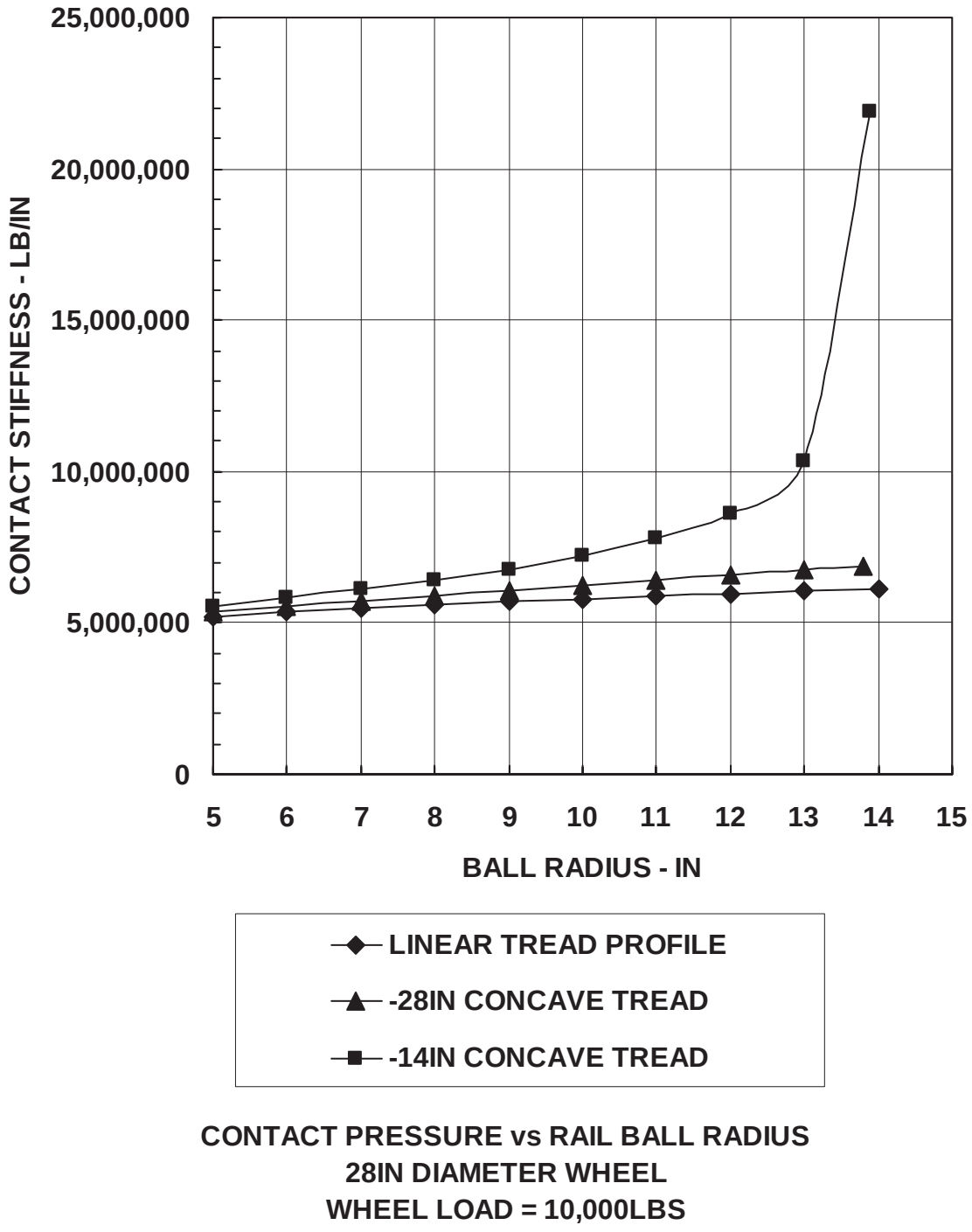

Figure 9.4.7 Variation of dynamic contact stiffness with rail head radius

Fully conformal contact is a condition that is frequently associated with rail corrugation, and the above results suggest that fully conformal contact would greatly exacerbate wheel/rail forces and

corrugation. The contact acts like a cushion at frequencies above perhaps several hundred Hertz, reducing dynamic contact stresses and noise. Increasing the contact stiffness by increasing wheel/rail conformal contact increases the high-frequency contact forces between the wheel and rail and thus increases wheel/rail noise.

Fully conformal contact should be prevented, at least on tangent track, to reduce wheel/rail dynamic forces, reduce corrugation rates, and reduce noise.

9.4.4 Residual Stress Accumulation—Shakedown

As noted above, the contact stresses induced during rolling contact under even modest loads are substantial. Residual stresses may be induced in the rail running surface during train operations, and these residual stresses protect the rail running surface from cyclic fatigue. This process is called "shakedown." Shakedown does not harden the rail, in the sense of resistance to wear, but simply moves the internal stress state by plastic deformation to a position where subsequent contact stresses do not produce plastic yield. This process is well described by K. L. Johnson.[72]

9.4.5 Work Hardening

Additional operation over long periods may work harden the rail. However, at rail transit loadings, work hardening also may not occur. While contact stresses may initially exceed the yield stress, shakedown stress accumulation may impede long-term work hardening. To the extent that a hardened running surface is desirable to reduce wear and corrugation rates, head-hardened or alloy rail should be procured.

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Chapter 10—Transit Signal Work

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CHAPTER 10—TRANSIT SIGNAL WORK

10.1 INTRODUCTION

The objective of this chapter is to provide trackwork designers, managers, inspectors, and contractors with a basic knowledge of the terminology, requirements, devices, and coordination issues for a rail transit signal system.

10.1.1 General

Typically, there are five types of light rail transit guideway configurations to be considered, each with different signaling characteristics:

1) In-Street Mixed Traffic Right-Of-Way (Streetcar)

   Street-running light rail systems can be operated without railway signals on a line-of-sight basis at speeds consistent with safe operation and traffic regulations. LRV operators generally must behave as if they were operating a motor vehicle, obeying the applicable traffic regulations and, of course, being observant of the locations and movements of motor vehicles and pedestrians.

   In this configuration, there will generally be no signaling systems other than traffic signals. The traffic signal system may be configured to provide the LRT preferential right-of-way access over cross traffic to improve speed by minimizing intersection delays. In such cases, the presence of the LRV would be detected by signal preemption devices such as overhead wire contactors, wheel detectors, induction couplers, or other vital or non-vital devices. “Priority” is more common than absolute preemption. This holds the light rail green phase longer or shortens the light rail red phase, but doesn’t give absolute preemption.

2) Semi-Exclusive Right-Of-Way

   This is a configuration where the LRT is within the street but train operation occurs in a dedicated semi-exclusive trackway, physically separated from other vehicle lanes by curbing or fencing except at intersections. However, train control circuitry may be provided to influence the operation of traffic, including special indications to control train movements at intersections, similar to the streetcar scenario above. Operation may still be on a line-of-sight basis, but signal systems may be provided to protect switch operations at junctions and crossovers. Access into the operating environment by motor vehicles or pedestrians is prohibited except at defined, controlled intersections. There is normally no broken rail detection where embedded track is used since derailment is unlikely with the pavement holding the rails in general alignment.

3) Exclusive Right-Of-Way

   Where higher speed train operation occurs in an exclusive right-of-way, trains use signal systems to avoid collisions with other trains and with street vehicles crossing the tracks.

   The principles of light rail transit signaling in an exclusive right-of-way are similar to railroad main line signaling methods of providing for the safe movement of trains. The track is divided

into segments called blocks. Signals keep two trains from occupying the same block at the same time and generally keep an empty block between trains that are travelling at the posted or indicated speed. Track circuits detect trains in a block. Block systems ensure train separation with safe stopping distance. Interlocked switches and crossovers protect against conflicting routes and improper switch operation. Transit signaling also provides block supervision as required for highway street operation, warning of approaching trains at grade crossings and supervising coordination with proximate vehicle traffic schemes as required for system performance and safety.

Common features in an exclusive right-of-way include the following:

- Power operation of track switch facing points: power on/off switches, time sequences, induction couplers, or other non-vital devices are used to improve LRV speed by eliminating stops to throw switches, thereby allowing trains to keep moving.
- Block supervision (single-track, low-speed operation): similar to preemptive devices, allows an opposing train to advance without incurring schedule delay, if possible.
- Block and switch protection: basic railroad signaling technology employing wayside signals, sometimes in conjunction with mechanical or inductive train stops, to provide safe operation (newer light transit systems have employed cab signals with or without train stops for continuous speed control or communication-based train control without train stops).

4) Highway Grade Crossings

Grade crossing warning: based on railroad signaling technology, gates and flashers generally eliminate any need for the LRV operator to slow down to determine if a grade crossing is clear. Grade crossing traffic control based on flashing lights only or even traffic signals only have been employed, but flashers with gates are generally recognized as the most effective type of crossing warning system. Crossing warning indicators are provided ahead of the highway grade crossing to inform the train operator as to the operational status of the highway crossing equipment.

5) Yard and Shop

Operational rules and small turnouts generally restrict train operations in yards to low speeds, typically 5 to 10 miles per hour. Busy switches are normally power operated; infrequently used switches are often hand thrown. Signals or indicators are provided where required and can be either integral with or external to the switch machine. Signal system architecture for a yard can be either vital or non-vital design and include track circuits.

The choice of which system is most appropriate for a specific section of track is based on operational and sometimes political considerations. A light rail system may utilize different signal technologies at different locations based on these concerns. A street-running operation at relatively low speeds requires different controls than a high-speed operation on an exclusive right-of-way.

10.1.2 LRT Operating Environment

Design differences in light rail systems are primarily related to their operating environments. Since the latter can vary over a large range, the appropriate level of signal automation varies by transit agency, their operating requirements for speed and headways, and the configuration and alignment of the track system components, including special trackwork. These can vary significantly along a given LRT line. As discussed in Chapter 1, it is not uncommon for an LRT line to be very much like a streetcar along one segment of its route, but have a semi-exclusive or completely exclusive trackway only a short distance further down the track. The optimum level of signal sophistication depends on such local circumstances and is generally determined by the transit agency responsible for providing the service.

While these issues have relatively little effect on trackwork, they have a significant effect on track alignment. Specifically, where the maximum diverging speed over turnouts or civil speed restrictions are enforced by the train control system, there are a limited number of speeds that can be enforced by the cab signal codes. The actual speed assigned to each code can vary from property to property, and a decision needs to be made early on in the track design as to what the enforced speeds should be. It does little good to design a curve to accommodate a civil speed limit of 45 mph [about 70 km/h] if the available speed commands are 30 mph and 50 mph [about 50 and 80 km/h]. Since the curve isn't good for 50 mph, it would be restricted to 30 mph by the train control system. In addition to the increased travel time, if the Ea for the curve was determined based on the unattainable 45 mph, Eu would be much less than planned and an overbalance situation might exist.

10.1.3 Transit Signal System Design

The system designer is obliged to consider the signaling technology available to provide the desired system operating performance at the least total cost. Within the scope of light rail transit applications, a well-established catalogue of proven technology is available.

Transit signal system design must consider not only what technology is available, but also the most rational combination of equipment for a particular application. Signal systems are customized or specified by each transit system to provide safe operation at an enhanced speed. The location of signal block boundaries is based on headway requirements and other considerations such as locations of station stops, terminals, highway crossings, storage tracks and special interlocking operating requirements.

Selection and spacing of track circuits for AC and DC propulsion systems are influenced by many factors. These include the degree of detection required for broken rails or defective insulated joints, the level of stray current control required, the frequency of interfering sources of power (propulsion and cab signaling), location of cross bonding, unbalance of track circuits, and the inherent advantages of various types of track circuits.

10.2 SIGNAL EQUIPMENT

10.2.1 Switch Machines

10.2.1.1 General

Track switches can be operated by hand or by power. When time and convenience are important, automated switch machines are advantageous. Switch machines may be controlled from a central control facility, central instrument house or signal hut/bungalow, or by the vehicle operator.

Switch machines are used on main lines, interlockings, and yards. Switch machines are used to operate switches in turnouts and crossovers in both main line interlockings and in yard tracks. In addition to switch rails, switch machines may also be used to operate movable point frogs, derails, wide-to-gauge derails, wheel crowder derails, or wheel stops. The type of switch machine selected is dependent on the type of track installation—timber or concrete switch ties, embedded track or direct fixation track, clearances, and the requisite operating parameters. For example, switch machines used in a yard are typically designed to be "trailable," permitting a trailing movement through a set of closed switch points without damage to the mechanism. This capability simplifies the operating circuitry.

10.2.1.2 Trackwork Requirements

Switch machines in open ballasted track rest on headblock switch ties and interface with turnouts through operating and switch rods. This interface is often complicated, particularly in direct fixation (DF) or embedded track, where blockouts in the concrete must be provided for proper clearance. The following elements associated with track and structure design should be considered when selecting turnout switch machines:

- Size of turnout or crossover
- Number of headblock ties (typically two)
- Size, height, width, spacing, and length of headblock ties
- Configuration of switch rods and accessories
- Thickness of number one rod
- Type of basket on number one rod
- Distance from centerline of switch machine to gauge line of the nearest rail
- Types of tie plate for number one and two ties
- Tie or mounting spacing between switch machine rods
- Type of derail or wheel stop or wheel crowder
- Location of mounting of switch machine, horizontally and vertically, relative to gauge line
- Insulation of trackwork switch, basket, and tie plate
- Switch throw distance
- Location of extension plate mounting holes and interface plate
- Lubrication of switch plate and track layout
- Rail section and weight
- Location of switch heater elements or type of switch heater (electric, hot air, or gas)
- Types of switch points
- Type of switch machine used in existing system—as related to spare parts and employee training for maintenance and adjustment
- Inserts installed in concrete switch ties for mounting bolts of the switch machine

10.2.1.3 Switch Machines

Track switches need some sort of mechanical device to change the orientation of the switch from the straight movement to the diverging movement and back and to also hold the switch points in the desired orientation. There are two broad categories of such devices:

- Switch stands, which are manually operated devices with no direct connection to the train control system. These are typically used only on seldom-used tracks that do not have any sort of signaling system. See Chapter 6 for additional discussion on switch stands.

- Switch machines, which are switch-throwing devices that do have a connection with a train control system. In most cases, these are power-operated devices although they usually incorporate a means of manual operation in the case of a system failure. These devices are discussed further in the paragraphs below.

Several terms are useful for understanding train movements through the switches of an ordinary turnout:

- A facing point movement is one heading in a direction from the switch point toward the frog, and a train can be taking either the straight track or making a diverging movement.

- A trailing point movement is one heading in a direction from the frog to the switch points with the tracks converging upon each other.

  - The “Normal” position of a switch is usually, but not always, with the switch set to accommodate a straight facing point movement.

  - The “Reverse” position of a switch is usually, but not always, with the switch set to accommodate a diverging facing point movement.

  - A Spring (or spring-back) switch is a switch that is ordinarily in one orientation but will permit a trailing point movement from the other track and automatically revert to the original normal position after the rail vehicle wheels have passed.

Spring switches are not normally installed in main track because the point rails cannot be locked in position. With no lock, either a switch malfunction or improper train movement might lead to derailment. Where spring switches are used, strict operating rules and signal systems to verify point closure are recommended.

10.2.1.3.1 Electric Switch Machines

Electric switch machines are common for light rail operations because of the ready availability of electric power throughout the system. Electric switch machines are rugged, reliable units designed for any installation where a reliable source of electric power is available. They are available in a variety of operating speeds and motor voltages. Electric switch machines may be used in main line and yard service. For installations where extra vertical clearance is needed for a third-rail shoe or vehicle clearance or on-board train control equipment, a low-profile electric switch machine can be used.

Switch machines are usually specified to meet the requirements of the AREMA *Communications & Signaling Manual*, Part 12.2.5, Load Curve Figure 1451-1 and thereby provide ample thrust to operate the heaviest of switches. Electric switch machines are normally provided with one throw rod, one lock rod, and one point detector rod connected to the rails. They are also available with two lock rods and two detector rods for use in cases where it might be appropriate to throw the switch rails independently. The track designer and signal designer must coordinate to ensure the specifications cover supply, installation, and adjustment of these critical elements. Gauge plate extensions can be supplied that attach the switch machine to the track switch to aid in holding the adjustments of the switch machine. Electric switch machines are usually installed adjacent to the normally closed point of the switch, so that the switch rods are in tension for the preponderance of train movements.

Since about 2000, electric switch machines that are entirely contained within a steel cross tie have become available. These require no headblock ties, switch rods, gauge plates, clips, basket, or lug. The in-tie electric switch machine is bolted to stock rail, but requires a reinforcing bar on the switch point. They can be advantageous in constrained areas. They greatly simplify ballast tamping; since there are no switch rods in the ballast cribs, an automatic tamper can work straight through the switch area.

10.2.1.3.2 Electro-Hydraulic Switch Machines

An electro-hydraulic switch machine can be suitable for use with virtually any type of switch point (tee rail or groove rail); wide or narrow point openings/switch throw; and standard, narrow, or broad track gauges. The switch machine can be installed at the side of the tracks or in the center of tracks, directly on ties or with rail-foot attachment.

10.2.1.3.3 Electro-Pneumatic Switch Machines

Electro-pneumatic switch machines require a reliable source of compressed air. While this is economical for heavy rail transit, which features short block lengths and frequent interlockings, the economics on light rail lines usually make air power switches too costly.

10.2.1.3.4 Hand-Operated Interlocked Switch Machines

Hand-operated interlocked switch machines are typically used where facing point lock protection is required to help safeguard the movement of high-speed main line traffic over a switch. These switch machines contain a locking bar that, with the switch in the normal position, enters a notch in the lock rod. This arrangement locks the switch points in their normal position to provide facing point lock protection. At layouts where additional train protection is required, a switch control controller box is installed with a point detector rod attached to the switch point.

10.2.1.3.5 Yard Switch Machines

Yard electric or electro-hydraulic switch machines are simple and compact machines designed for installation in tight spaces. A common yard movement will see a light rail vehicle making a converging movement at a turnout with the switch set contrary to the desired path. Doing this with a standard main line design switch machine would wreck the mechanism. Because of this, some yard electric switch machines can handle trailing moves at maximum speeds up to 20 mph [30 km/h] (which is faster than the speed limit in most transit yards) without damage to the locking mechanism. If point detection is required, an additional circuit controller can be installed. Typical yard switch machines can be adjusted for throw, from 4.5 inches [114 mm] up to 5.5 inches [140

mm] for tee rail and 2.4 inches [60 mm] to 4 inches [100 mm] for grooved rail. The low-profile yard electric switch is available with external switch indicator lights.

Electro-pneumatic switch machines are also available for yard applications, and a compressed air plant at the yard or maintenance facility may make them economical.

10.2.1.3.6 Embedded Switch Machines

Embedded (surface) switch machines are designed to throw tongue and mate, double-tongue, or flexive tongue switches. The embedded switch machine can be installed between the rails (preferred) or on the outside of the switch tongue on a paved street. Embedded switch machines can be powered from the overhead catenary (at 600 to 750 volts DC) or from a separate AC or DC source. Embedded switch machines may be all-electric operation or employ electrical systems only to control hydraulic mechanisms and power a hydraulic pump.

Most embedded switch machines permit the switch to be trailed without damage to the machine; however, routine use of this feature typically causes wear on mating parts and increasing difficulty in keeping the switch machine in proper adjustment. Impressing upon operations personnel that the trailable feature is not intended for routine use can be an appreciable challenge for the transit agency's track and signal maintainers.

There are no North American standard specifications applicable to embedded switch machines. Because they are virtually never used in railroad applications, they are not addressed by the AREMA *Communications & Signaling Manual*. While there are some applicable requirements in European standards such as BOStrab, EN, IEC, and VDV requirements, they generally apply to the interfaces and not to the machines themselves. These machines are therefore generally designed in accordance with manufacturer's specifications, and there are appreciable differences between makes and models.

Because embedded switch machines are usually used with tongue switches, their range of adjustment is appreciably different from switch machines designed for open track. Typically, machines designed for application to flexive tongue switches of European design can be adjusted only within a range of 60 to 100 mm [2.4 to 4 inches]. Caution is advised when applying these machines to AREMA-style split switches. If the throw (measured at the switch rods) is set too low, there may be insufficient clearance between the stock rail and the switch point at the rear of the side planing on the switch rail. See Chapter 6 for additional discussion of this issue.

Like all power switch machines, the switch case for an embedded switch machine must be grounded for the safety of maintenance personnel. In the case of switch machines that are energized by a tap to the traction power, this can create a stray current leak unless detailed correctly. The responsibility for that design should be with the signal system designers, but the trackwork designer needs to verify that it has been done so the trackwork isn't blamed for a stray current leak.

Drainage of embedded switches and switch machines is critical. The embedded switch machine track box should be drained to a nearby storm pipe, because an undrained box collects a mixture of sand, water, salt, etc., that increases wear on moving parts and prevents their proper

lubrication. An access/cleanout box is often installed on the field side of the switch casting to provide access to connecting rod adjusting nuts if they extend beyond the switch.

10.2.2 Impedance Bonds

10.2.2.1 General

Impedance bonds are necessary when insulated track joints are used to electrically isolate track circuits from each other and broken rail detection is desired for both rails. The impedance bonds permit propulsion current to flow around the insulated joints while inhibiting the flow of signal current between adjacent track circuits. They also provide crossbonding between the two rails of the track so as to balance propulsion current for track circuits and touch potential and provide negative return to the traction power substation.

The stagger between insulated joints should be 2 feet [0.6 meter] or less for transit signaling to reduce the amount of cable needed as well as the unbalance in the current in the rails associated with impedance bonds. The usual location for impedance bonds is in the gauge of the track; some railways place them on independent structures outside of the vehicle dynamic envelope.

Audio frequency track circuits are separated from each other by using a different frequency in each circuit; thus, they do not normally require insulated joints to isolate the track circuits, and impedance bonds are generally unnecessary. The exception occurs when insulated joints are used with audio frequency track circuits to provide a precise definition of block limits, such as at signal locations.

Where broken rail detection is not required, a single rail track circuit can be used and all of the return traction current will flow through the continuous return rail. With single rail track circuits, an insulated joint is used in the non-return rail, which is then called the signal rail.

Impedance bond connections to the rails can be by various methods. See Article 10.2.11 for additional information on all types of cable connections to the rails.

10.2.2.2 Trackwork Impedance Bond Requirements

The following elements associated with track and structure design should be considered when designing impedance bonds:

- Tie spacing for impedance bond equipment
- Location of tie or direct fixation mounting holes for signal equipment
- Location of impedance bond, either between or outside the rails
- Location of guard and restraining rails
- Location and spacing of insulated joints
- Space for cables and conduit to pass beneath the rail
- Conduit and cable location for signal equipment
- Block-out requirements for embedded or direct fixation
- Mounting to wooden, concrete ties and direction fixation

10.2.2.3 Types of Impedance Bonds

10.2.2.3.1 Audio Frequency

Audio frequency impedance bonds are designed to terminate each end of audio frequency track circuits in transit installations. They provide the following:

- Low resistance for equalizing propulsion current in the rails
- Means of cross bonding between tracks to reduce electrical resistance along the negative return path
- Connection for traction power negative return
- Means of coupling the track circuit transmitter and receiver to the rails
- Means of coupling cab signal energy to the rails
- Means of inhibiting the transmission of other frequencies along the rail

10.2.2.3.2 Power Frequency

Power frequency impedance bonds are designed for use in AC or DC propulsion systems that use insulated joints to isolate track circuit signaling current from signaling currents of adjacent circuits, but permit propulsion current to flow around the joints to or from adjacent track circuits. AC impedance bonds are usually rated for 300 amps per rail and DC impedance bonds are usually rated for between 1,000 and 2,500 amps per rail. Typically, power frequency impedance bonds are installed in pairs at insulated joint locations and mounted between the rails across two adjacent ties, as an impedance bond is required on each side of the insulated joints.

Power frequency impedance bonds provide the following:

- Low resistance for equalizing propulsion current in the rails
- Means of cross bonding between tracks to reduce electrical resistance along the negative return path
- Connection for negative return
- Means of coupling the track circuit transmitter and receiver to the rails
- Means of coupling cab signal energy to the rails
- Means of inhibiting the transmission of other frequencies along the rail

10.2.3 Loops and Transponders

10.2.3.1 General

Loops and transponders are used to transmit information to the train independent of track circuits. They may be found in all types of trackwork and can be used for intermittent transmission or continuous control systems. In determining the type or location of loops or transponders to be used for a light rail transit system, consideration should be given to the operation plan, type of track circuits, propulsion system, and train control system that is installed.

10.2.3.2 Trackwork Requirements

The following elements associated with track and structure design should be considered when designing loops or transponders:

- Location of loop or transponder inside or outside the rail
- Mounting the loop or transponder to the ties and direct fixation trackway
- Tie spacing and mounting method for loop or transponder

- Cable and conduit location for signal equipment
- Block-out area for loop or transponder and junction box

10.2.3.3 Types of Loops and Transponders

10.2.3.3.1 Speed Command

Speed command loops are used to provide a means for coupling cab signal energy to the rails. Typically, speed command inductive loops are installed with or without rubber hoses within the turnout diverging track. They may be attached to the tie or concrete or clipped to the rail. Normally, a rubber hose with wire inside is installed near the inside of the rail at interlockings and turnout switches. These loops provide isolation from the track circuits.

10.2.3.3.2 Daily Safety Test

Daily safety test loops are used to provide a means for verifying the function of carborne train control equipment. Typically, daily safety test inductive loops are installed with or without rubber hoses in the yard tracks or mainline storage tracks in the field. They may be attached to the tie or concrete or clipped to the rail. Normally, a rubber hose with wire inside is installed near the inside of the rail. Newer LRT systems do not require daily safety test loops due to on-board testing on the LRT vehicle.

10.2.3.3.3 Train Location/Train-to-Wayside Communication

Train location loops or train-to-wayside (TWC) communication loops are designed to provide very precise definition of a train's location, automatic vehicle identification, and either one-way or two-way train/wayside communication. A wire loop installed between the rails and on ties links the train to the rails. The horizontal loop of the wire is directly mounted or placed in a heavy polyvinyl chloride (PVC), epoxy, or fiberglass (FRE) conduit or channel that may also be encased in pavement. These loops can vary from 8 feet to 20 feet [2.5 to 6 meters] depending on the operation.

10.2.3.3.4 Traffic Interface

Loops or transponders can be used to pre-empt traffic signals or provide phasing command and release of traffic control devices.

10.2.3.3.5 Continuous Train Control Loop

Typically loops between stations are transposed at regular intervals. This provides a signal to the on-board equipment that can be used to recalibrate an on-board odometer. In station areas, short loops may be provided for accurate automatic station stopping.

10.2.3.3.6 Transponders

Transponders are designed to transfer data, such as the intended train routing or destination, between the vehicle and train control equipment that is located along the trackway. Transponders or antennae may be mounted overhead or on the wayside, embedded between the rails, or mounted to concrete surfaces or ties.

10.2.4 Wheel Detectors/Axle Counters

10.2.4.1 General

Wheel detectors and axle counters are used to detect trains without relying on a track circuit. Since they do not require insulated joints, they cause less interference with traction return current than detection devices that depend on electrical signals in the rails. When used without track circuits or cab signaling within the rails, they eliminate the need for insulating switch rods. However, they are unable to detect broken rails. In selecting the type and model of wheel detector/axle counter, consideration should be given to the operation and mounting method used.

10.2.4.2 Trackwork Requirements

The following elements associated with track and structure design should be considered when designing wheel detectors/axle counters:

- Type and size of rail
- Mounting hole size
- Conduit and cable location
- Rail grinding
- Maintenance
- Block-out requirements or box requirement
- Drainage for embedded housing

10.2.4.3 Types of Wheel Detectors/Axle Counters

The wheel detector/axle counter unit consists of a detector head or mechanical detector arm, mounting hardware, a logic board, and interconnecting cabling. Wheel detectors and axle counters are mounted with clamps that attach to the base of the rail or are bolted directly to the web. The wheel detectors/axle counters are activated when a vehicle passes. The magnetic wheel detector/axle counter is independent of the wheel load and subjected to almost no wear since there is no mechanical interaction between the detector and vehicle wheels.

10.2.5 Train Stops

10.2.5.1 General

Train stops (also known as “trip stops”) activate a train’s braking mechanism if the train passes either a restrictive cab signal aspect or a fixed signal. They can be inductive units or electrically driven mechanical units. The latter are found only on older installations and are ill-suited for areas where snow and ice can interfere with their operation. In designing train stops, consideration should be given to the location of vehicle equipment, type of trackbed, operation (directional or bi-directional), relationship to wayside signal layouts, and location of the train stop elements. Train stops are used in exclusive rights-of-way and are not applicable to mixed traffic street-running applications.

10.2.5.2 Trackwork Requirements

The following elements associated with track and structure design should be considered when designing train stops:

- Type of track—ballasted, direct fixation, or dual block
- Tie spacing
- Type of tie—timber or concrete

- Location of train stop
- Conduit and cable location
- Relationship to signals, insulated joints, and impedance bonds
- Location of train stop and wheel detectors relative to top of rail

10.2.5.3 Types of Train Stops

10.2.5.3.1 Inductive

Inductive train stops are designed with a magnetic system that interacts with carborne vehicle control equipment. Both the vehicle magnet and the track magnet need to be strategically mounted on the vehicle and track, respectively.

10.2.5.3.2 Mechanical (Electric and Pneumatic)

As noted previously, mechanical train stops are generally considered obsolete technology, but are found on older systems. The key component of the mechanical train stop is the driving arm, which is pulled to the clear position a ½ inch [12 mm] below the top of the running rail by either an air or electric motor and returned to its tripping position by a spring. Mechanical train stops are usually mounted on plates midway between two rails with the operative arm outside either the right-hand or the left-hand rail.

10.2.6 Switch Circuit Controller/Electric Lock

10.2.6.1 General

A switch circuit controller is a mechanism that provides an open or closed circuit indication for a two-position track appliance, such as a switch point. A mechanical linkage to the crank arm of the controller actuates its normal/reverse contacts. The switch circuit controller provides independent contacts that allow separate adjustments at each end of the stroke. Commonly used to detect switch positions, the switch circuit controller can be used to detect positions of derails, bridge locks, slide detectors, and tunnel doors. The switch circuit controller can shunt track circuits as well as control relay circuits. Electric switch locks prevent unauthorized operation of switch stands, hand-throw switch machines, derails, and other devices. In determining the rods and type of switch circuit controller/electric locks, consideration should be given to operation, type of switch or derail, mounting, and clearances. The switch circuit also protects vehicles by ensuring that the switch points are closed.

10.2.6.2 Trackwork Requirements

The following elements associated with track and structure design should be considered when designing wheel switch circuit controller/electric locks:

- Type of track bed—ballasted, direct fixation, or dual block
- Type of tie—timber or concrete
- Length of tie
- Left- or right-hand layout
- Type of hand-operated switch machine or derail
- Number and location of connection lugs on derail
- Location of conduit and cable
- Dapping of tie

10.2.6.3 Types of Switch Circuit Controller/Electric Lock

10.2.6.3.1 Switch Circuit Controller

A switch circuit controller is a ruggedly constructed unit commonly used with switches to detect the position of switch point rails. The switch circuit controller has a low clearance profile and is mounted on a single tie.

10.2.6.3.2 Electric Lock

An electric switch lock operates by a means of a plunger that is lowered into a hole in the lock rod connected to switch points, derails, or other devices. Another electric switch lock secures the hand-throw lever on a switch stand or switch machine in the normal position. This type of electric switch lock does not require dapping the ties and may be applied to either a right- or left-hand layout.

10.2.7 Signals

10.2.7.1 General

Wayside track signals are usually light fixtures either mounted on poles or positioned at ground level (dwarf signals) next to switches. Some applications in embedded track have used the equivalent of airport runway lights mounted in the pavement between the rails. Several variations of color-light signals with various indications are currently in use on light rail systems. In determining the type and configuration of wayside signals to be used, consideration should be given to operation, clearances, signal layout, track layout, right-of-way, and insulated joint locations.

10.2.7.2 Trackwork Requirements

The following elements associated with track and structure design should be considered when designing signal mast installations:

- Insulated joint locations
- Right-of-way clearances
- Conduit and cable location
- Vehicle clearances
- Stopping distances
- Sight lines for vehicle operators
- Block out for indicators, embedded track
- Drainage for signal indicator box

10.2.7.3 Types of Signals

Long-range color-light signals consist of one or more light units with an 8.4-inch [213-mm] outer lens for high signals and a 6.4-inch [162-mm] lens for dwarf (low) signals. These high and dwarf signals can be equipped with lenses for improved visibility along curved tracks. Dwarf signals are designed for direct mounting on a ground-level pad such as a concrete foundation. The main line high signals have backgrounds, hoods, pipe posts, ladders, pole mounting brackets, and foundations.

Compact color-light signals designed specifically for use in close-clearance transit tunnels are also made. A 5-inch [127-mm] lens signal is typically used in subway installations where space is limited, while a 6.4-inch [162-millimeter] lens signal is also available for outdoor service. Transit signals are supplied with brackets for mounting on subway walls, ceilings, or poles. These signals are available as incandescent or LED types.

Embedded signals or indicators are designed to be installed in embedded track. The housing of the embedded signal can require drainage.

Yard signals or indicators are fabricated in dwarf assembly case aluminum or steel housing. These signals or indicators are available as incandescent or LED types.

10.2.7.4 Signal Locations

Trackway civil design needs to provide sufficient space for signals and other types of train control system infrastructure. This includes not only space for the signal itself, but also sufficient working space so that signal maintainers can safely inspect and work on the equipment. Vehicular access to these locations is also highly desirable as the maintainer's tools and equipment can be heavy and bulky.

Signals are normally installed to the train operator's right side when operating in the normal direction for the track. They are ordinarily located at a minimum clearance offset distance from the dynamic envelope. For convenience of field measurements, the offset dimension is frequently specified from the gauge line of the near rail instead of the centerline of track. Where insulated joints are used in territory with double rail track circuits, the signal is typically located between the two insulated joints. The signal can be moved ahead of the insulated joints to a distance no greater than the overhang of the vehicle.

Signals and switch position indicators for embedded tracks are sometimes installed in low-profile "runway lights" between the rails of the embedded track. The housing of these embedded signals can require drainage.

Yard signals or indicators are installed next to the switch machine side or in front of the switch machine or at the insulated joint location to a junction box or pedestal.

10.2.8 Bootleg Risers/Junction Boxes

10.2.8.1 General

Bootleg risers/junction boxes provide a central termination point for signal cables. Bootleg risers/junction boxes come in a variety of sizes, with or without pedestals, and are constructed of cast iron or steel. Based on the application of the bootleg risers/junction boxes, the location can be in the center of tracks, outside or inside the gauge side of the running rail, outside the end of tie, outside the toe of ballast, or next to the switch machine or other signal appliance. In selecting the type and size of bootleg risers/junction boxes, consideration should be given to the type of trackbed, cable, signal equipment, and mounting method used.

10.2.8.2 Trackwork Requirements

When designing bootleg risers/junction boxes, the following elements associated with track and structure design should be considered:

- Conduit and cable location
- Type of trackbed—ballast, direct fixation, embedded, or dual block
- Tie spacing
- Maintenance
- Stray current control

- Drainage track junction boxes for embedded track
- Block-out requirements
- Rail size and type

10.2.8.3 Types of Bootleg Risers/Junction Boxes

10.2.8.3.1 Junction Boxes

Pedestal-mounted junction boxes are typically used in ballasted track at switch machines, switch circuit controllers, track circuit locations, etc., as a central termination point for underground cables. A variety of adapter plates allow the junction box to be used with air hose adapters and connectors during testing and when staging the installation of cabling that cannot be completed due to incomplete trackwork construction.

In direct fixation track or dual-block tie track, the junction box enclosure is mounted to the invert of the track structure or on the wall of the structure.

10.2.8.3.2 Rail Junction Boxes

The rail junction box is a compact enclosure for providing access to cable connections to the rail in embedded track. It could be made of either fabricated steel or a casting. The rail junction box is bolted to the web of the rail, most frequently on the field side of the rail. Stainless steel or brass connector bolts are inserted through the track rail junction box wall from the inside and are attached to the web of rail. To minimize stray current potentials, the connector bolts are insulated in the rail junction box wall. These units will normally have drainage outlets since the lids will not be watertight.

10.2.8.3.3 Bootleg Risers

Bootleg risers are designed as a termination point between the underground cable and the track wire to the rail or signal device. They are available with a bottom outlet, as well as side and bottom cable outlets. A typical bootleg riser installation in ballasted track would locate the riser box either in the center of the track with the top slightly below the top of ties or just beyond the end of ties.

10.2.9 Switch and Train Stop Heaters/Snow Melters

10.2.9.1 General

Switch and train stop heating systems are designed to keep rail switches, switch rods and tongues, and mechanical train stop arms free of ice and snow in a predictable and reliable fashion. In designing the heating system, consideration should be given to the type of power available, type of trackwork, type of trackbed, operation, type of train stop, type of switch machine, and mounting method used.

10.2.9.2 Trackwork Requirements

When designing switch heaters and snow melters, the following elements associated with track and structure design should be considered:

- Size of turnout or crossover
- Length and other dimensional details of the switch rails
- Ease of maintenance or replacement of heater parts
- Type of rail brace with notch, if required

- Conduit and cable location
- Junction box(es) location(s)
- Length of switch point
- Number of switch rods
- Type of trackbed—ballasted, embedded, direct fixation, or dual block
- Stray current control requirements
- Access to switch tubular heater
- Block out for embedded track and support

10.2.9.3 Types of Switch/Train Stop Snow Melters

There are several snow melter systems commonly used in the transit industry. The most popular system for open track features tubular resistor electric snow melters that can be installed on either the field side or gauge side and either at the underside of the rail head or at the base of the rail. For gauge side installation, holes for heater support clips are drilled in the neutral axis of the rail using a clearance drill with $^{3}/_{8}$-inch [10-mm] bolts. For field side installation, snap-on clamps are used (no drilling is necessary). Tubular electric snow melters mounted on the field side of the rail require the special trackwork rail brace to be notched for passage of the snow melter.

The rail web heater can also be used to prevent switches from freezing. The rail web heater is a low-density panel that spans the rail web. It consumes 20 to 40% less power than a tubular heater installation. Rail web heaters are interconnected to provide more heat to the point and snapped into place using rugged clips and a special clip tool. No braces need to be loosened or grooved to allow installation, which provides for easy removal in the spring prior to track maintenance or repair.

Power is supplied to electric snow melters from the overhead catenary through a snow melter control cabinet or case or from a local AC power source.

Switch rod heaters are used to melt snow and ice away from switch rods. These switch rod heaters are installed in the bottom of the crib where the switch rods are located. They consist of a steel channel or panel with tubular electric heaters or a series of heating elements attached. The tubular electric heater can be mounted on a swing bracket that clamps to the base of the rail on the field side and is adjustable for all sizes of rails.

For heating tongue switches used with grooved rail, the tubular electric heater can be installed either beneath the switch tongue bed or outside of the switch housing. Wherever it is positioned, the heating element needs to be well-protected from the grit that is endemic in any street environment, yet still be easily accessible for inspection and replacement. While positioning the heater tube beneath the tongue bed would seem to be the most efficient way to get the heat to the location where it is needed, access for heater tube renewals could be very difficult. To provide access for maintenance, the tubular electric heater can be installed in a steel tube attached to the outside of the switch housing. The installation detail needs to avoid the possibility of heat damaging any electrical and acoustical isolation systems that are part of the tongue switch assembly and naturally should not create a stray current leakage path.

Train stop mechanisms in open track can be furnished with hairpin-shaped heaters or heating panels.

Other types of snow melting systems include oil, natural gas, or an electric high-pressure heating unit that forces hot air throughout the switch area via ducts and nozzles. An alternate snow blower arrangement uses ambient non-heated air to blow snow clear of the switch point areas.

10.2.10 Highway Crossing Warning Systems

10.2.10.1 General

Highway crossing warning systems provide indications to motorists that a light rail vehicle is approaching the crossing. Such systems are commonly but erroneously called crossing "protection" systems. That term is incorrect as there is no way to protect the train from a motor vehicle whose operator elects to ignore the signals and no way to protect the motorist from the consequences of his/her failure to comply with the signals provided. What the signals can do is warn the operator of a motor vehicle on an intersecting path that a train is approaching. Currently, there is no effective way to advise an LRV operator that motor vehicles are approaching a crossing, making it highly desirable to keep the "sight triangles" at all four quadrants clear of obstructions such as buildings and vegetation.

The most common configuration for highway crossing warning systems is conventional flashing light signals, either with or without gates, such as those commonly used on freight railroad grade crossings. In determining the type and configuration of the highway crossing warning system, consideration should be given to LRV operations, type of track circuit, roadway layout and posted speeds, traffic signal(s) location, right-of-way, and clearances. The challenge of fail-safe crossing warning systems is to separate the LRV and highway traffic without closing the crossing to motor vehicle traffic for extended periods of time. The Federal Highway Administration's *Manual of Uniform Traffic Control Devices* (MUTCD) now includes recommendations for at-grade crossings of LRT tracks. These requirements are included in Part 8 of the MUTCD (2009 edition), which can be downloaded from the FHWA's website.

In some cases, crossing warning systems will include specific signs and warning signals for pedestrians. The track designer needs to coordinate the layout of the crossing surface and the approach pavements so that pedestrians are directed along paths where they can clearly see warning devices. In addition to the requirements currently in the MUTCD, there have been numerous experimental installations of barriers and crossing warning devices to promote the safety of both pedestrians and motorists and, by extension, the operators and passengers of the rail vehicles. One example is active signage that flashes to indicate a "Second Train Coming" from the opposite direction of the one that initially activated the warning system. This issue is dynamic and LRT design teams are encouraged to consult recent trade publications and published papers for state-of-the-art information.

Crossing warning installations should be interconnected with any traffic signals located within 200 feet [60 meters] of the highway grade crossing. Additional advance warning of approaching LRVs, i.e., more time than required at an ordinary crossing, may be required for proper operation of the traffic signals so as to "flush" certain legs of the intersection prior to train arrival. An on-site diagnostic team meeting is usually required at an early date to discuss all of the implications of the warning system being proposed.

Crossing warning systems work best, with minimal delays to traffic, when the approach time of trains to the crossing can be accurately predicted. If train speed might vary, or if there is a possibility that the train might even stop on either the approach or departure side of the crossing, vehicular traffic could be significantly delayed for reasons the motorist will not understand. This can lead to motorist disrespect for the crossing warning devices and unsafe behavior by drivers. Such problems can occur whenever either wayside signals or stations are located too close to the crossing. Ideally, such items should be located either well in advance of the crossing warning system start circuits or sufficiently beyond the departure end of the crossing to ensure that the longest train will have cleared the street and the signal circuits prior to stopping.

10.2.10.2 Trackwork Requirements

When designing highway crossing warning systems, the following elements associated with civil and trackwork design should be considered:

- Location of insulated joints (if required)
- Location of crossing slabs
- Minimum ballast electrical resistance
- Tie spacing
- Right-of-way clearance to highway crossing equipment
- Conduit and cable location
- Insulation of running rails from each other if a track circuit is used for the warning system
- Drainage of trackbed, roadway, and sidewalks so that the ballast section is kept clean and dry
- Sight triangle clearances

10.2.10.3 Types of Highway Crossing Warning Systems

A typical highway crossing may consist of flashing light units, gate mechanisms with arms up to 38 feet [11.6 meters] long, poles, foundations, cantilever assemblies, cables, case or signal houses, junction boxes, and track circuits with island circuits. Some LRT highway crossing systems have a highway crossing system warning indicator signal (typically a “lunar light”) in advance of the crossing to verify the operational status of the highway crossing warning equipment to the LRV operator.

Where the LRT is located in a street with exclusive lanes and train speeds do not exceed 25 mph, it is usually more appropriate to control the crossings with vehicular traffic by using conventional traffic signal technology as opposed to railroad-type flashing lights and gates. Special signal heads, frequently using different colors or symbols than ordinary traffic lights, provide the LRV operator with guidance as to when he may safely cross the intersection. Crossings with train speeds between 25 and 35 mph should be carefully evaluated to determine whether traffic signals will provide the sufficient degree of safety. Crossings with train speeds higher than 35 mph should use railroad-type warning systems, although, as of 2010, this topic is under study by TRB and others. Reference to the current edition of MUTCD is essential.

10.2.11 Signal and Power Bonding

10.2.11.1 General

Signal and power bonding is used to establish electrical continuity and conductive capacity for traction power return and signal track circuits. It prevents the accumulation of static charges that

could produce electromagnetic interference or constitute a shock hazard to track maintenance personnel. It also provides a homogeneous and stable ground plane, as well as a fault current return path.

Power bonding is typically installed at all non-insulated rail joints, frogs, restraining rails, guard rails, and special trackwork locations. Power bonding of the restraining rails requires special attention to avoid creating run-around paths that can falsely energize the track circuit or compromise broken rail detection.

There are basically two types of rail connections used in the transit industry: mechanical (electric compress, bolted, or drill pin) and exothermic welding. In determining the type and the amount of signal and power bonding, consideration should be given to type of track circuits, capacity of the traction power equipment, type of rail, vehicle wheels, and the degree of desired broken rail detection.

10.2.11.2 Trackwork Requirements

The following interface elements associated with track and structure design should be considered when designing signal and power bonding:

- Type and size of rail
- Spaces for bonding to be installed
- Space for signal and power bond passing beneath the rail
- Type of trackbed—ballasted, direct fixation, embedded, or dual block
- Location of rail joints, insulated or non-insulated
- Location of guard and restraining rail
- Signal cable connection to rail in special trackwork
- Location of crossbonding and traction power negative return
- Block outs for power cables for connection or junction box
- Location of signal track fouling wires

10.2.11.3 Types of Signal and Power Bonding

Impedance bond leads are factory made to system specifications and impedance bond type for ease of installation, eliminating a typically cumbersome field application. One method of connecting cables to rails is via plug bonds. This method involves drilling a hole in the rail and hammering the plug into the hole. Exothermic welding, on the other hand, generates molten copper to create a solid bond between the cable and rail or between cables.

Bonds used to be welded to the rail by either electric arc welding or gas welding, but these methods are seldom used anymore because of the amount of equipment that needs to be mobilized and the need to provide a skilled welder. Arc or gas welding to the base of the rail used to be common, but it is not recommended due to the high chance of thermal damage to the rail.

Rail bonding connections in embedded track require special consideration in part because they will be completely concealed by the pavement. It is therefore recommended that the track details provide for both the protection of the cable connection to the rail and permit inspection and maintenance to occur. A common detail is to provide fabricated steel boxes where traction power cables (for example, those going back to the traction power substation) connect to the

rails. The conduits carrying the cables will stub up into the box, which then has a lid held in place by corrosion- and tamper-resistant fastenings.

Traction power bonding cables are sometimes run completely around any bolted special trackwork unit so that negative return is not dependent on a series of joint bonds within the unit. This can enhance the safety of workers that might be replacing a component such as a frog. Coordination with signal design is essential.

Volume 3 of *TCRP Report 71* evaluated broken rail problems that some rail transit systems were having in the vicinity of large cable bonds that had been attached to rails by the exothermic process. The general conclusion of that study was that large bond cables should be attached to the rail (particularly high-strength alloy rails) by the exothermic process only under rigorous procedures with close attention to quality control. In the absence of such controls, there is a high probability that the rail steel in the attachment zone will be transformed into an untempered martensite and thereby be subject to brittle fracture. Hence, unless exothermic bonding quality processes can be ensured, plug bond systems or bolted or compression electrical connections are the preferred method for attaching large cables to the rails.

Advantages of exothermic welding versus plug bonds or bolted or compressed connections include the following:

- The installation resistance of a length of exothermic weld bond is less than that of other types of bonds of the same length and cable stranding. Resistance will not change throughout the life of the bond.
- Provided the rail was ground clean prior to making the weld, there should be no corrosion between an exothermic weld bond and the rail. Intermittent signal failures due to the varying resistance of a corroded rail joint will be eliminated.
- Because bonding does not rely on a friction fit, bond losses caused by dragging equipment, reballasting, and snowplows are reduced.
- Train traffic will not loosen a properly installed exothermic weld bond.
- Rail head signal bonds that are applied within 5 inches [125 mm] of the end of rail (per AREMA *Communications & Signaling Manual*, 8.6.40 C.1) maximize detection of a broken rail compared to plug bonds that are applied outside of the splice bars.
- Rail web bonds from 0.2 to 0.4 square inches [140 to 250 square mm] provide a convenient means of connecting all cable outside the confines of the splice bar, including special trackwork.
- The exothermic weld process provides an efficient field method for any electrical connection from signal and power to ground.
- Properly applied, the exothermic weld normally outlives the conductor itself.
- No hole drilling is needed, and there is no chance of an oversized hole providing an inadequate connection.

Advantages of plug bonds, bolted bonds, or compression bonds versus exothermic welding include the following:

- The rail connector clamp can connect cables from 0.4 to 1.6 square inches [250 to 1000 square mm] to the running rails.

- Mechanical connectors provide a rail connection without the risk of overheating the rail steel during installation.
- Rail connection can be easily relocated or temporarily removed without grinding the rail or chopping the connection.
- Proprietary plug-type rail bonding systems are available that have a greatly reduced probability of loosening under vibration compared with much older designs of bonds that are merely hammered into drilled holes in the rail web.
- Unlike exothermic welds, plug bonds, bolted bonds, and compression bonds can be installed in the rain.
- Splice bar to rail web bonds may be used to detect a break in the splice bar itself.
- Where signal bonds cannot be installed from the field side due to tight areas, such as frogs and switches, a multi-purpose bond can be used by drilling through the rail web.
- They facilitate testing because their removal and reinstallation is far easier than cutting and splicing an exothermically welded bond.

10.3 EXTERNAL WIRE AND CABLE

10.3.1 General

Various types of cable and methods of installation are required for transit signal systems. Main cables are those cables that run between housings or that contain conductors for more than one system function. Local distribution cables are those cables running between housing and an individual unit of equipment. In selecting the method of installation of external wire and cable, consideration should be given to cost, maintenance, and type of right-of-way.

10.3.2 Trackwork Requirements

When determining the location of external wire and cable the following should be considered:

- Conduit and cable location
- Maintenance of trackwork
- Drainage because surface-mounted cable troughs can obstruct the flow of surface water, so provision must be made for storm water to flow around or under the cable trays
- Locations of pull boxes, handholes, manholes, duct banks, etc.
- Compaction of soil and subballast
- Location of cable trough
- Visual impact
- Staging of construction so that ballast is not fouled with subgrade fines from cable excavation

10.3.3 Types of External Wire and Cable Installations

10.3.3.1 Cable Trough

A cable trough system is a modular surface trench that protects and provides continuous accessibility to the signal cables. Signal cables can exit and enter the cable trough system either from the bottom or sides. Trough segments are typically installed in the ballast, often just beyond the ends of the cross ties. Where the cable needs to cross the track, the cable trough is often laid

in the crib between two cross ties. Cable trough may be in locations where maintenance trucks might drive over it and hence should be capable of supporting an H-20 highway load at any point.

The typical cable trough installation requires a trench of minimum width and depth to provide free access to the top of the trough while maintaining crushed stone alongside and below the trough so as to provide free passage of storm water below. Leveling blocks are placed in the bottom of the trench to keep trough segments properly aligned. The maximum stone particle size beneath the trough should not exceed ¾ inch [19 mm]. Fill material should not be placed on frozen ground and should be tamped. The cable trough should be placed so that the uppermost part is 1 inch [25 mm] higher than the surrounding ground or ballast surface. Where cable trough is located within the ballast section, its installation typically occurs after the track construction is completed and the ballast is fully dressed. It is important that the cable trough installer restores the ballast section to its original configuration and does not foul the ballast stone with unsuitable materials.

Signal maintainers like cable trough since it provides easy access to the entire length of the cable, making it much easier to troubleshoot faults compared to cables that are either in a duct bank or strung on overhead poles. Track maintainers generally dislike cable trough because it can interfere with track maintenance activities, particularly cross tie replacement and ballast tamping. Where cable trough is located in a ballast crib, it is typically necessary to use manual methods of ballast tamping on the adjacent cross ties.

10.3.3.2 Duct Bank

The underground duct system can be completely encased in concrete with a minimum clearance of 2 inches [50 mm] between conduits and a minimum cover of 3 inches [75 mm] to the outside edges of the concrete. If a non-metallic conduit is not encased in concrete, allow 18 inches [460 mm] of separation for signal cables carrying 0 to 600 volts from low voltage cables. For cables carrying over 600 volts, non-shielded cables should be installed in rigid metal conduits with a minimum cover of 6 inches [150 mm]. For cables carrying over 600 volts in rigid non-metallic conduits, the conduit should be encased in no less than 3 inches [75 mm] of concrete or have 18 inches [450 mm] of cover if not encased in concrete. Cables are connected to the duct bank systems using handholes, pull boxes, and manholes for proper pulling points or cable routing. A minimum cover of 30 inches [760 mm] is recommended for protection (per AREMA *Communications & Signaling Manual,* 10.4.1.F.1 and F.2) when signal cables pass under tracks, ballast, or a roadway.

Ideally, all underground ducts will be installed prior to installation of the subballast so that the latter is not disturbed by subsequent excavation. However, this sequence results in conduit stub-ups into the track area that must be protected during placement of the subballast and subsequent track construction activities. It is important that the responsibility for the care of these duct bank risers be assigned in the contract documents. Deferring the construction of the risers until after the track is installed is not a total solution. At a minimum, specifications should require the conduit installer to avoid fouling the ballast and subballast with excavated material and to replace the excavated material with identical granular fill and ballast complying with the original specifications. Regardless of construction sequence, close coordination between designers and between constructors is essential.

10.3.3.3 Conduit

Encased or direct burial conduit should be installed as outlined above or as required by the National Electric Code, Article 300-5 and 1110-4(b). Normally, conduits are installed with not less than 30 inches between the top of the conduit and finished grade.

10.3.3.4 Direct Burial

Signal cable and wire should be buried to a uniform depth where practicable, but not less than 30 inches [760 mm] below finished grade. Where signal cable and wire is installed within 10 feet [3 meters] of the centerline of any track, the top of the cable should be a minimum of 30 inches [760 mm] below the subballast grade.

Signal cables and wires should be laid loosely in the trench on a sand bed a minimum of 4 inches [100 mm] thick and covered with a minimum of 4 inches [100 mm] of sand before backfilling. Backfill should be compacted to not less than 95% of the maximum dry density of the respective materials, as determined by AASHTO Test Designation T-99, or to the original density of compaction of the area, whichever is greater.

Where direct burial signal wires cross the tracks, it is beneficial to install the wiring prior to the tracks. This improves the integrity of the track structure, but complicates signal installation. Any trenching that must be done anywhere in the completed ballast section must avoid fouling the ballast with excavated spoil. Backfilling must use proper ballast and subballast materials and be thoroughly compacted. It often will be necessary to re-tamp the ballast not only over the trench but also for several ties on each side on the disturbance.

Signal cables can be plowed in at a depth of 30 inches [760 mm] and 12 inches [300 mm] beyond the toe of subballast. Avoiding the track ballast and subballast is important to maintaining the structural integrity of the track.

10.4 SIGNAL INTERFACE

10.4.1 Signal/Trackwork Interface

Signaling and trackwork interface issues include the following:

- Location of insulated joints
- Location and mounting requirements for impedance bonds, train stops, track transformers, junction boxes, and bootleg risers
- Physical connection of impedance bond track cables and track circuit wiring
- Location and mounting layout of track switch operating mechanisms and switch machine surface and subsurface areas (ballast, direct fixation, and embedded)
- Cable and conduit requirements for interconnection of signal apparatus at track
- Location and installation of train stops, inductive loops, transponders, wheel detectors, and axle counters
- Interface pick-up with the traffic signal system
- Location of block outs and foundations for wayside signal equipment
- Electromagnetic interference/electromagnetic compatibility (EMI/EMC)
- In cab-signaled territory, coordination of track alignment civil speeds with available cab signal speed commands

- Grounding
- Yard signaling
- Grade crossing warning systems
- Wayside equipment housings and cases
- Corrosion control
- Cross tie spacing for signal control equipment, impedance bonds, train stops, and switches
- Tie size and length requirements for switch machines and derails
- Signal cable connection to rail at special trackwork
- Physical connection of switch machines to special trackwork, with adjustment and testing
- Loop or transponder mounting on track for train-to-wayside communication
- Spaces for cables and conduit passing beneath the rail
- Location of guard and restraining rails with respect to insulated rail joints
- Horizontal clearance between track and wayside signals and equipment
- Vertical clearance between track and signal equipment
- Space and drainage for switch machine in direct fixation or embedded track
- Provision for installation of snow melters
- Location of switch indicators for embedded track
- Location of cross bonding and negative return cables
- Location of speed limits
- Ballast resistance
- Foundations for cases, housings, pushbuttons, and signals

10.4.2 Signal-Station Interface

The following signal equipment is typically installed at station locations: impedance bonds, inductive loops, bootleg risers, junction boxes, and transponders. If the station is located near an interlocking or highway crossing, there should be sufficient room from the end of the platform to the signal equipment (impedance bonds and signals) and insulated joints if required. This may affect the site layout of the station and might control the position of the platform with respect to the crossing.

10.4.3 Signal-Turnout/Interlocking Interface

The following signal equipment can typically be found at turnouts and interlockings:

- Switch machines
- Impedance bonds
- Inductive loops including speed command loops
- Train stops
- Bootleg risers
- Junction boxes
- Switch controllers
- Electric locks
- Transponders
- Wire and cables
- Signal and power bonding

- Cases/signal equipment houses (“bungalows”)
- Signals
- Snow melter systems

The design of the track circuit and fouling protection used will determine the location of insulated joints in the special trackwork. Typically in transit applications, the insulated joint should be located approximately 23 to 25 feet [7 to 7.6 meters] ahead of the switch points to allow for the use of plug rails for bonded insulated joints. The size of the turnouts and crossovers determines the speed at which the train can operate. This speed should be one of the available cab speeds. The insulated joint for the turnout fouling must be located with a minimum of clearance, taking into account the longest overhang from the front of the train to the first axle of any equipment that may operate on the track. Location of insulated joints at turnouts and interlockings needs to consider the distance from the signal or point of switch to ensure switch detector locking. This distance can vary from 25 feet to 50 feet.

10.5 CORROSION CONTROL

Leakage of stray currents into the ballast bed and earth can be a significant problem if the cables running from the rails are electrically connected to the impedance bond housing case and the case is in contact with the earth. This can occur if the cases are mounted on reinforced concrete where the mounting bolts contact the re-bar, if the bottom of the case is resting on concrete, or if dirt and debris accumulate between the bottom of the case or signal equipment and the concrete. An accumulation of ballast, dirt, or other debris around the locations where the cases or signal equipment are installed along the right-of-way can also provide a path for current leakage. This type of installation can result in a continuous maintenance problem if an effectively high rail-to-earth resistance cannot be achieved.

Some impedance bonds are located outside the tracks on timber ties to eliminate points of possible contact with earth. The center taps of the impedance bonds should be insulated from the mounting case. Switch machines also must be electrically isolated from the running rails.

Any embedded switch machines and junction boxes need to be insulated from the rail and insulated from concrete or other material.

Dissimilar metals shall be isolated or separated for corrosion control. Materials selection needs to consider the environmental conditions where the items are being installed.

Yard tracks should be isolated from the main line tracks to reduce corrosion. For additional information on corrosion control, refer to Chapter 8.

10.6 SIGNAL TESTS

10.6.1 Switch Machine Wiring and Adjustment Tests

Switch machine wiring and adjustment tests verify the wiring and adjustment of the switch machine. These tests should preferably be carried out, in conjunction with the track installer, to

confirm throw rod capability, ensure point closure, and ensure proper nesting of the switch point rail to stock rail.

10.6.2 Switch Machine Appurtenance Test

Switch machine appurtenance tests verify the integrity of switch machine layout by taking resistance measurements across the following assemblies:

- Center insulation of the front rod
- Front rod to switch point
- No. 1 vertical or horizontal switch rod center insulation
- Throw rod insulated from No. 1 switch rod
- Point detector piece insulated from switch point
- Lock rod insulated from front rod
- Other vertical rods as required per layout
- Switch machine insulated from the running rails
- Switch machine extension plate insulated from the running rails

10.6.3 Insulated Joint Test

Insulated joint tests measure the resistance between two ends of the rail separated by insulating material. An insulated joint checker requires the traction power system to be disconnected. Any reading less than 30 ohms should be evaluated. Measurements for a set of insulated joints should be within 30% of each other or they should be rechecked. Insulated rail joint tests for AC track circuits can be performed using a volt-ohm-meter.

10.6.4 Impedance Bonding Resistance Test

Impedance bonding resistance tests, using a low-resistance ohm-meter, ensure that a proper connection has been made.

10.6.5 Power and Signal Bonding Test

The power and signal bonding test is to ensure that the resistance across the rail connection is not greater than recommended by AREMA *Communications & Signaling Manual*, Part 8.1.30 and 8.1.31.

10.6.6 Negative Return Bonding Test

Negative return bonding tests verify the resistance of each mechanical or welded power bond using a low-resistance ohm-meter.

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Chapter 11—Transit Traction Power

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CHAPTER 11—TRANSIT TRACTION POWER

11.1 GENERAL

Light rail systems, as defined in Chapter 1, use electrical power from overhead wires to provide traction power to the light rail vehicles. The rails, sometimes in conjunction with supplemental negative return cables, act as the return conductor to the negative terminal of the rectifiers. Therefore, the electrical properties of the rails and tracks require special consideration. To obtain good conductivity for the track as a whole, a rail system must have a low resistance not only for reasons of economy but also for safety. This requires a low voltage drop in the rails over the length of the track structure.

11.1.1 Traction Power System Components

The complete traction power system consists of the following:

- Traction power substation (TPSS) that converts commercial alternating current electricity into the direct current power used by the light rail vehicles.
- Cables connecting this substation to the wayside distribution system.
- Wayside distribution system providing adequate current at appropriate voltage levels throughout the alignment. The principal element of the wayside distribution system is the overhead contact system (OCS), more commonly called the “catenary.” In some cases, there will be supplemental cables running parallel to the route to “feed” additional power to the contact wire.
- The rails, which carry the negative return current from the LRV back to the vicinity of the substation. In some cases, these will be supplemented by negative return feeder cables.
- Return system cables connecting the running rails to the substation.
- In some cases, a system of corrosion control drainage cables to collect stray traction power current and take it back to the appropriate substation. These corrosion control drainage cables are separate from and not to be confused with negative return feeder cables.

11.1.2 Traction Power/Track Interfaces

There are four elements in the traction power system that affect, or are affected by, track alignment and trackwork design and the construction and maintenance of track systems:

- Traction power positive supply system, including substation locations
- Wayside catenary distribution positive system, providing power to the vehicles
- Traction power negative return through the rails
- Corrosion control measures to minimize the level and effects of stray currents on adjacent conduits, pipes, and cables

11.2 TRACTION POWER SUBSTATIONS

Traction power substations take commercial alternating current power from the local utility company and convert it into the direct current required by the LRVs. The optimal locations for the traction power substations are determined using a computer model that simulates proposed LRT operations along an accurate geometrical and geographical depiction of the planned route. The model will include not only the horizontal and vertical alignment of the track, but also the achievable design speed so as to determine the power demand of the LRT system during peak periods. Therefore, in the early stages of any light rail transit project, track and traction power designers must interface to integrate the traction power system into the overall system design.

The final selection of substation sites is an iterative process with repeated simulations to confirm the capability of the traction power system to sustain peak-hour operations. The sequence of events to develop substation sites is as follows:

- The traction power designer, using the simulation program, selects theoretically ideal TPSS positions along the route, taking into account the distribution system's voltage drop and the lowest voltage acceptable to the vehicle without degrading performance. The normal, single contingency criterion for determining traction power system sufficiency is to test the system with alternate substations out of operation and verify whether an acceptable level of LRT operations can be sustained.
- The designer discusses these proposed locations with the local power utility to determine any impacts of the proposed power demand on their network. The utility then evaluates the availability of power circuits and the potential impacts on its other customers.
- An agreement is eventually reached, if necessary, by moving the substation to enable it to be supplied from lightly loaded power circuits or by building spur cables to the substation location. It is also important, for reliability, that the power company avoid supplying two adjacent substations from the same circuit.
- It is not always possible to position the traction power substations in the optimal location, particularly in urban areas where available sites may be limited by many issues, including political realities.

After an agreement is reached with the power company, the traction power designer can finalize the substation design. While the TPSS can be a constructed building into which equipment is installed, most substations for new and renovated light rail systems are modular, factory-assembled units that are delivered to the site complete. They are erected on a prepared base that incorporates an extensive grounding network below the concrete. These modular units are more economical than constructed buildings. Depending on the neighborhood where they are sited, modular TPSS units are sometimes screened by landscaping or architectural walls.

Substations are located along the route as close to the tracks as possible within the constraints of available real estate. However, the final placement must also consider interfaces and underground cable duct routes for the power distribution supply and return systems, access roadways, and security requirements. The impact of this construction on trackwork design is limited to the interfaces with the supply and return power distribution system.

The electrical sectionalization of the distribution system usually takes place at the substation for all travel directions. Placement of a substation at, or near, a crossover is often desired to sectionalize electrical supply for each travel direction and to optimize the operational flexibility of the track system.

11.3 WAYSIDE DISTRIBUTION SYSTEM

Broadly speaking, wayside distribution systems can be subdivided into the overhead contact system, which is discussed in Article 11.4, and supplemental cabling systems to connect the catenary to the traction power substations. The latter is the topic of this article.

Each TPSS is typically linked to the trackway by underground conduits. One set of conduits runs to and up one or more catenary system poles to carry the cables that provide power to the catenary. The conduit risers can be located either on the outside surfaces of the OCS poles or within the poles, either of which can require an appreciable foundation at trackside. Once the power supply is terminated to the catenary, the positive side of the traction power supply distribution usually remains on aerial structures and does not interface further with the track.

Another set of conduits and cables runs to the track and provides the negative return path for traction power back to the TPSS. The design of the trackbed needs to accommodate these traction power supply system conduits. Adequate space is required beneath the track for the conduit systems (including terminations), conduit risers, and manholes. The track itself must accommodate connections of the negative return cables.

If the overhead contact wire system is a single filament trolley wire, it is usually necessary to have supplemental feeder cables as well so that the overall traction power distribution system has sufficient electrical capacity to provide current without unacceptably large voltage drops. In urban areas, these feeder cables are, for aesthetic reasons, most often routed through underground duct banks that run parallel to the tracks. The feeder cable must be periodically connected to the trolley wire, usually at every third to fifth OCS pole. At each such location, a manhole will be located along the trunk duct bank and a branch conduit will run over to the OCS poles. The overall design of the trackway must accommodate these ducts and manholes.

A messenger/feeder can be placed above the trolley to obviate the need for parallel feeders. The messenger-to-trolley vertical dimension (construction depth) can be made small (6” to 12”) to reduce visual impact, with no effect on track design.

Less often, supplemental feeder cables are carried on the OCS poles rather than being routed in underground conduits. Many legacy streetcar lines used this configuration. This substantially reduces the impacts on the trackway design, but the overhead cables negate some of the visual aesthetic benefits of a single filament trolley wire system as it effectively just moves the catenary messenger cable from a position directly over the track to a location along the poles.

11.4 CATENARY SYSTEMS

11.4.1 Introduction

The OCS on a light rail system usually consists of a simple catenary system that incorporates both a messenger cable from which a contact wire (also known as a trolley wire) is suspended. This configuration is both electrically efficient and economical to construct. The word "catenary" is actually a mathematical term that describes the curve assumed by a flexible cable that is suspended at its ends. It therefore can technically be applied to virtually all types of OCS. However, it is usually intended to mean an OCS where a messenger cable supports a simple trolley wire with both conductors being used to carry the power used by the light rail vehicles.

In visually sensitive areas, a single trolley wire may be utilized so as to minimize the number of wires above the tracks. This is a common requirement where the tracks are in city streets and the light rail line has the characteristics of a streetcar.

The style of catenary and most of the basic design parameters can be developed prior to finalization of the track configuration. However, the application of a catenary design to suit the track layout can only proceed after the track alignment has been finalized.

11.4.2 Catenary Alternatives

The track alignment engineer needs to understand what type of OCS is proposed so that adequate clearances can be provided along the trackway for poles, pole foundations, and associated hardware. There are generally three styles of OCS used on LRT systems: simple catenary, low-profile catenary, and single filament trolley wire system. All types of OCS can have either of the following configurations:

- Fixed terminations at the end of each wire run, causing the conductors to either sag or rise as the temperature varies, or

- Balanced weights at one or both ends of each wire run so as to maintain constant tension and height regardless of the climatic conditions of the project site.

Fixed termination OCS typically requires heavier poles, larger pole foundations, and more robust line hardware than balance weight design because of the higher tensile loads that occur in the wires during cold weather. Therefore, modern, lightweight catenary systems almost always use balance weight tensioning to limit the load applied to the poles. Regardless of whether balance weight or fixed termination design is used, the catenary is typically separated into 1-mile segments. The conductors are overlapped at the ends of these segments so as to provide a smooth passage of the vehicle pantograph from one segment to the other. The track alignment design may need to accommodate additional poles at these overlaps.

The details of the OCS will also vary with the type of current collector used on the light rail vehicle. Pantograph current collectors can use either a fixed termination or balance weight OCS as the pantograph head can easily sweep over an overlap between one run of trolley wire and the next. Vehicles equipped with a trolley pole generally require a fixed termination system since the

trolley wire running surface must be continuous, without any gaps or overlaps. In addition, trolley hardware for pole operation is not normally suited for pantograph operation.

A simple catenary system uses a messenger wire to support the horizontal trolley wire. Both conductors are used to transmit power from the substation to the vehicle. In a simple catenary system, the system height at the supporting poles—the distance between the contact or trolley wire and the messenger—is approximately 4 feet [1.2 meters]. In tangent track, this allows spans between poles of up to 240 feet [73 meters].

The low-profile catenary system is similar to the simple catenary design, except that the system height at the support is reduced to approximately 18 inches [457 millimeters] and sometimes less. This style is often applied in aesthetically sensitive areas where a lower profile and simple single-wire cross spans are more desirable. The trade-off, however, is that the span length between supporting poles is reduced to approximately 150 feet [46 meters].

Single filament trolley wire systems, which were traditionally used on legacy streetcar lines, are considered by many persons, and particularly by lay audiences, to be visually less obtrusive in the urban environment. It provides power through a single trolley wire that must be supported at least every 100 feet [30 meters]. The span length is limited by the sag of the unsupported trolley wire, which in high temperatures could otherwise fall below the minimum elevation required by the National Electrical Safety Code. It is also limited by the structural capacity of the supporting hardware to carry the weight of the entire length of a span between supports. Single filament trolley wire also usually requires the wire to be supplemented electrically by parallel feeders that must be frequently connected to the trolley wire to maintain sufficient voltage for LRV operation. These feeders may either run underground through a series of ducts and manholes, which are expensive, or be hung from poles. The latter position can arguably be just as visually intrusive as the messenger wire in a catenary system and merely relocated to a different point in the observer's line of sight. The overhead wire design to accommodate trolley pole operation requires more support and registration points and can therefore have nearly twice the number of poles than the equivalent simple catenary system.

11.5 CATENARY DESIGN

11.5.1 Introduction

Generally, technical papers have not addressed rail/catenary interface issues since transit catenary design has developed from operating railway systems where the track is already in place and the catenary must follow the existing track layout. For new light rail transit lines, while it's possible to consider OCS design during the early planning stages of the project, unfortunately, such is rarely the case. In many new transit systems, the track alignment has been selected prior to the catenary designer's involvement in the project. The results of this lack of coordination are chronicled in *TCRP Report 7: Reducing the Visual Impact of Overhead Contact Systems*. Involving the catenary designer during the route selection and conceptual track alignment design stage can be cost-effective in the long run and reduce the visual impact of the catenary system.

Horizontal and vertical track alignment, trackwork, passenger station locations, substation sites, etc., must all be determined before the preliminary catenary engineering can proceed. However,

the locations and design of these components can greatly influence the catenary design and its visual impact on the environment.

11.5.2 Conceptual Stage

The catenary engineer's task is to develop a conductor configuration to supply power to the vehicle from a position over the track that will allow good current collection under all adverse-weather, operating, and maintenance conditions. The engineer must develop the most economic solution considering the aesthetic constraints set by the community. This task involves resolving the number of wires in the air with the number of poles, supports, and foundations to achieve an efficient and environmentally acceptable design.

The catenary system is the most conspicuous and arguably the most visually undesirable infrastructure element of a light rail transit system. *TCRP Report 7* discusses "visual pollution," to the extent that it cited a case where a community refused to introduce an electric-powered transit system because of the expected visual impact. However, with rare exceptions, overhead wires are needed to distribute power to light rail vehicles. Therefore, poles are needed to support and register them over the pantograph under all adverse conditions. However, if the track alignment designer considers the catenary constraints, then the size and number of poles can be minimized.

The catenary distribution system interfaces with trackwork in the following manner:

- On single-wire catenary systems, the track designer must accommodate the longitudinal and transverse track feeder conduits that support the electrical distribution system.
- The track designer must also provide adequate clearance between tracks for foundations, poles, catenary balance weights, guy anchors, and guy cables.
- Track design and maintenance standards must be coordinated so that the vehicle pantograph remains beneath the catenary wires under all adverse operating and climatic conditions.

11.5.3 Application of the Catenary System to the Track Layout

Since the wire runs in straight lines between support points and the track is curved, pole layout is a compromise between the number of poles and the requirement that the contact wire remain on the pantograph under all adverse climatic, operating, and maintenance conditions. Even though the pantograph head can be up to 6.5 feet [2 meters] wide, allowances for track alignment, gauge, cross-level tolerances, vehicle displacement, roll, pantograph sway, and pole deflection means that only its central 18 to 24 inches [460 to 610 millimeters] are actually available for the wire to sweep across the pantograph head. At the midpoint between supports, this distance is reduced to near zero due to deflection of the wires under maximum wind and ice loading conditions.

The allocation of pole positions must take into account the limitations of the catenary style, the profile of the contact wire necessary to accommodate overhead bridges and grade crossings, track curvature, crossovers and turnouts, underground utilities, etc. Therefore, if the track is designed with the catenary constraints in mind, economies can be achieved. The following paragraphs identify parameters that should be considered by the track designer.

11.5.3.1 Track Centers

The clearance between poles and the track is defined by the system's dynamic clearance envelope, which comprises three elements: the vehicle dynamic envelope, construction and maintenance tolerances, and running clearances. Therefore, if center poles with supporting cantilevers on each side are desired to reduce cost and visual intrusion, then the distance between tracks should allow for this envelope from each track plus at least 12 inches [305 millimeters] to permit installation of standard-sized poles.

11.5.3.2 Horizontal Curves

If the track is tangent, there will be no track-related constraints other than right-of-way boundaries when placing the poles along track. However, as the wire negotiates curves using a series of chords, the number of supports is very dependent on the curvature. Therefore, as with other light rail system components, minimization of curvature and avoidance of extremely tight curves is most desirable in catenary system design. This can be a challenge for in-street LRT where even on a nominally straight street the tracks may need to frequently shift laterally to stay in a consistent traffic lane while dodging around left turn lanes. See Chapter 12 for additional discussion on this issue.

11.5.3.3 Vertical Profile

The minimum clearances between the underside of the contact wire and the top of rail are dictated by the National Electrical Safety Code. In exclusive guideways, the usual requirement is a clearance of 16 feet under any condition of loading, including wind, snow, and ice—although lower elevations are possible. Where the OCS passes over a public street, the minimum requirement is typically 18 feet. Whenever the height of the contact wire changes, the gradient of the wire relative to the track profile will be restricted, depending on the desired train speed. The track alignment engineer must therefore closely consider any locations where the track and catenary pass beneath a low-clearance bridge shortly after passing over a public road. If the relative gradient of the trolley wire to the track is too steep or the change in grade too great, it may require a speed restriction, which could then affect other issues on the LRT project.

11.5.3.4 Vertical Curves

Vertical curves become critical when in the vicinity of reduced-clearance overhead bridges. The rise and fall (sag) of the catenary messenger is governed by the formula:

$$\frac{WL^2}{2T}$$

where W is the weight of the catenary
L is the distance between supports
T is the tension in the messenger

Therefore, if there is an abrupt change in track profile near an overhead bridge, the track designer should consult with the catenary designer to ensure that the wire can negotiate the vertical curvature. This can be an issue when the LRT crosses a street at grade where, per the NESC, the wire must be high and then must immediately pass beneath a low-clearance bridge. Extreme cases can require some ingenious interdisciplinary coordination.

11.5.3.5 Interlockings

The catenary/pantograph interface is a dynamic system. There are certain constraints applied to ensure that the system operates efficiently under all speed and weather conditions. The pole positions at turnouts are tied to the point of intersection (PI) of each turnout. It is desirable for the distance between the inner crossovers of a universal interlocking (i.e., two independent crossovers, one right hand and one left hand) to be approximately the same length as each crossover (PI to PI).

Double, or "scissors," crossovers can be wired; however, they present many difficulties for the catenary designer. Usually, for maintenance purposes, the inbound and outbound tracks are separated into different electrical sections. With tracks crossing within roughly 6 feet [2 meters], very limited space is available to insert a section insulator in the contact wire and still avoid the horns of the pantograph head. This is particularly difficult in higher speed sections using constant tension catenary design since the movement of wires along the track due to temperature change can aggravate the problem. By contrast, a single run of catenary can cover both crossovers placed end to end in a universal interlocking, making for much less costly OCS construction. Therefore, scissors interlockings should be avoided when catenary is employed.

11.5.3.6 Track Adjacent to Stations

Architecturally, the introduction of the catenary system is obtrusive. Architectural design tends to dictate the position of poles to suit the architectural theme within the station area. This impacts catenary pole positions adjacent to the station area, requiring close coordination between the architect and track and catenary designers to ensure adequate space for poles at stations and approaches.

11.6 TRACTION POWER RETURN SYSTEM

The train control signaling system, which often requires insulated joints in the rails, complicates matters of traction negative return, which wants the rail to be electrically continuous. The paragraphs that follow explain the basics of the track engineer's role in accommodating the conflicting needs of the signaling and traction power systems.

11.6.1 Territory with Two-Rail Track Circuits for Signaling

The traction power return system directly impacts track design. The traction power return system uses the running rails as an electrical conductor to "return" the traction power to the substation from which it originated. Traction power supplied to the train enters the running rail through the vehicle wheels and is extracted from the rail through impedance bonds in cables installed at each substation. Therefore, track designers must allow for impedance bond installation along with the associated conduit stub-ups and negative cabling at each substation. Where there is more than one track, in addition to the impedance bonds at each substation, impedance cross bonds are located along the track every 610 meters (2,000 feet) or less to equalize the traction return currents in the rails. At these locations, conduit stub-ups will be installed beneath the tracks connecting the two track directions. Impedance bonds are also required by the signal system at the end of each signal block.

11.6.2 Territory with Single-Rail Track Circuits for Signaling

Although most track circuits for signaling in new light rail systems are of the two-rail type, single-rail signaling track circuits do exist in older systems. In such systems, one rail is used for traction return and the other is designated the signal rail. This type of installation requires insulated joints separating the track circuits. With single-rail track circuits, the impedance bonds described in Article 11.6.1 are not required. The cross bonding provided between the traction return rails of separate tracks uses cables without impedance bonds for this purpose. Except for these differences, the same cabling is required between the traction return rail and substations as described in Article 11.6.1.

11.6.3 Territory without Signaling Track Circuits

The requirements for traction return in this type of territory are similar to those described in Section 11.6.1, except that no impedance bonds are required. Instead, cables are installed directly to the rails for both traction return at the substation and for cross bonding between the rails.

11.6.4 Rail Conductivity

Concern is occasionally voiced over whether the rails themselves have sufficient conductivity to carry the return traction power current. Well-meaning persons will sometimes suggest that the chemistry of the rail should be revised so as to enhance its conductivity. Such concerns are generally ill founded since rail of normal size (such as 115 RE) with normal rail chemistry, already has far more current-carrying capacity than the OCS. Moreover, the chemistry of rail, whether it is rolled in accordance with the AREMA specifications or the European Norms, has been carefully determined to produce rail with optimal wear and toughness characteristics. Modifying that rail chemistry in the pursuit of enhanced conductivity is extremely likely to result in rail of inferior mechanical properties. Such rail would likely not be guaranteed by the rolling mill. Further, unless the purchaser is buying several heats of steel rails, the steel companies will be unreceptive to interrupting their normal production methods.

11.7 CORROSION CONTROL MEASURES

In designing DC traction power systems, it is common and desirable to isolate and insulate the running rails from ground as much as possible. These issues are discussed at length in Chapters 4 and 8.

The traction power return system interfaces with trackwork in the following manner:

- The siting of impedance bond positions and cross bonds to adjacent tracks must be coordinated.
- The selection of rail insulation for tie plates and fastening clips suitable for track and traction power requirements must be agreed to by all parties.
- Continuity bonds on jointed rails must also be coordinated.

- The track designer and construction inspector should ensure that ballast is clear of rails so that return currents do not stray into the ground and cause corrosion problems in underground pipes and cables.
- Special consideration must be given to selecting the insulation of the rails at grade crossing and embedded track sections to ensure minimum leakage to ground.

11.8 MAINTENANCE FACILITY YARD AND SHOP BUILDING

The traction power system in the maintenance facility yard and shop area is usually different from and totally isolated from that used on the main line. This is because the yard and shop complex, including all of the infrastructure and underground utilities within it, is typically completely new. Therefore, the adverse effects of stray currents can be accounted for and mitigated in its design. For example, all underground utilities can be constructed using non-conductive or insulated pipes that will not conduct stray currents. Also, the traction power return current can be more easily controlled in a yard by increasing the quantity and locations of return cables. Because of these stray current mitigation measures, the trackwork insulation systems provided for the yard tracks could, in theory, be somewhat less robust than those used along main line track.

For this reason, yard tracks on light rail systems constructed up through the 1990s were often constructed with timber ties without insulated rail fastenings. However, since then, the prices of concrete cross ties and timber cross ties have gotten closer. More projects now seem to be using concrete cross ties with insulated rail fastenings in their yards as a matter of course. This trend has seemingly expanded from plain track to special trackwork zones as well, with insulated rail fastenings being applied to switch timbers and the adoption of concrete cross ties for yard tracks as well as main lines. While providing this level of electrical insulation within an insular yard facility is perhaps unnecessary from the perspective of stray current control, the simplicity of using the same track materials systemwide has much to commend it. In addition, yard tracks can be very difficult areas in which to change out defective timber crossties, so the increased service life of concrete cross ties can usually be justified by a life cycle cost analysis.

The grounding systems for the yard and main line must be electrically separate. This is achieved by inserting insulated rail joints in the yard entry track at each arrival and departure connection. The track alignment designer must carefully coordinate the locations of these insulated joints so that standing trains waiting to either leave or enter the yard do not straddle the joints for more than a few seconds. This can require an iterative coordination process including the track designer, the operations planners, and the traction power engineers.

In the maintenance facility building, the rails are installed directly into the shop floor system and are rigorously electrically grounded for safety of the personnel working on the vehicles. The return system is designed for current to return directly to the substation through cables to ensure that there is no potential difference between the vehicle and the ground. Space for the conduit and cables connecting each track section to the building substation must be coordinated. The shop tracks also contain insulated joints that electrically separate these totally grounded tracks from the yard track system. Usually, those joints are located in the ballasted track immediately beyond a concrete apron that typically is positioned along the face of the shop building so as to provide rubber-tired access to each shop doorway.

Yard track designers must still consider and account for the many conduit risers necessary to feed the numerous electrical sections in the overhead contact system. Extra coordination in yard areas should take place due to the additional users and electrical connections in the complex track layout.

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Chapter 12—LRT Track in Mixed Traffic

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CHAPTER 12—LRT TRACK IN MIXED TRAFFIC

12.1 INTRODUCTION

While it is highly desirable that light rail transit lines have an exclusive or semi-exclusive guideway, there frequently are circumstances where the only practical or affordable alignment is to place the tracks within a public roadway in a mixed traffic environment. This is, of course, the sort of guideway configuration used by traditional streetcar lines since the 19th century. In the case of a modern streetcar project in an historic district, the project definition might actually stipulate mixed traffic operation because of some desired ambiance.

Where these rail cars are operated in general traffic lanes, they are mixed with steerable, rubber-tired vehicles and should, to the greatest extent feasible, flow with traffic. They should not make any unusual or unexpected lateral movements, but rather function like the rubber-tired vehicles also using the street. However, there is a fundamental difference between rail cars and motor vehicles. Rail car operators can control the acceleration, deceleration, and speed of their vehicle so as to match the pace of traffic, but they cannot steer. Vehicle guidance is done by the tracks. This fundamental principle, which is well understood by the track designers, will not necessarily be so obvious to the civil/highway designers and the traffic engineers with whom they must interface. Collectively, the team must make certain that the goals mentioned above are achieved.

Trackwork within streets creates significant discontinuities in the pavement surface by replacing longitudinal segments of conventional paving materials with steel rails plus adjacent open flangeways. To the maximum degree practical, these steel surfaces and openings should not create hazards to other users of the street, including pedestrians and rubber-tired vehicles of all types.

Fitting trackage into locations where the street geometry imposes restrictions is challenging. Designers of the track, roadway, overhead contact system, and light rail vehicle all need to work closely with each other as well as with the project staff preparing the transit operations plan.

These issues, which meld roadway, traffic, and track engineering, need to be resolved as early as possible in the design process. The designers in each discipline should work in concert as the conceptual design matures.

12.2 TRACK POSITION WITHIN LANES

Typically, lane widths on urban streets are between 10 and 11 feet [3.0 and 3.3 meters] wide, occasionally narrower or wider. There are a number of factors that affect where, within these lanes, the LRT tracks might be placed. These include the actual lane width available, the dynamic width of the light rail vehicle, the presence or absence of station stops, and the presence of adjoining parking lanes. Occasionally, underground utility features that cannot be relocated may also constrain the position of the tracks.

12.2.1 Vehicle Width

There is no standard streetcar or LRV body width, but it will always be less, usually by about 2 feet [0.61 meters], than the width of any lane in which a track is laid. This width difference offers some design flexibility in that the centerline of the track does not necessarily have to coincide with that of the traffic lane so long as the dynamic envelope of the rail car does not encroach into an adjacent travel lane. Moreover, there are good reasons to consider offsetting these two lines.

12.2.2 Transverse Position within Lanes

Since the coefficient of friction between rubber and steel is lower than that of rubber on paving, especially when the roadway is wet, it is generally better for rubber-tired vehicles to travel with all tires on paving, rather than on steel rails. Offsetting the centerlines of the track and traffic lane facilitates this goal. The direction of the offset is immaterial; the intended effect is achieved either way. However, site-specific conditions (e.g., all stations are on the right side of the track) might indicate a preference for a particular juxtaposition of the two centerlines.

12.2.3 Adjacent Parking Lanes

Situations in which a parking or curb loading lane is positioned alongside of the shared lane utilized by LRVs must be carefully considered. Typically, these lanes are no wider than 8 feet [2.44 m]—a dimension sufficient for an ordinary automobile, but not for a wide delivery truck. These trucks can have body widths of 8½ feet [2.59 meters], plus mirrors, and will thereby frequently overhang the line, even if the truck driver is diligent about getting the tires close to the curb.

In such situations, if the position of the adjoining LRT track is biased toward the parking lane, it is likely that badly parked delivery trucks will foul not only the dynamic envelope of the LRV but perhaps even its static outline. In the latter case, if the truck driver cannot be found quickly, light rail service might be blocked for an extended period until the offending motor vehicle can be moved. In addition, since the LRV would be stopped in a general traffic lane, ordinary motor vehicle traffic could be impacted as well. Such situations may require that the parking lane width be larger than normal or that the track is biased away from the parking lane, or both.

12.3 ON-STREET STATION/STOP PLATFORMS

If the design calls for the station platforms to be in the form of a sidewalk bulb-out, a new problem is created. Since ADAAG (and similar regulations in other countries) stipulates that the platform edge must fall within 3 inches [76 mm] of the threshold of the LRV doors, the platform edge could encroach a substantial distance into the traffic lane, where it would be a hazard to motorists. Obviously, this encroachment into the traffic lane is extremely undesirable. One well-known modern streetcar line has dozens of examples of this situation, and the leading edge of very nearly every platform is marred by impacts from rubber tires because some motorists realized too late that the platform encroached into their path.

Note how this encroachment would exist even when the track is biased toward the platform side since the dynamic envelope of the LRV, which ordinarily should not be permitted to overhang the edge of the lane, is appreciably wider than the static envelope, which must match the platform. A

compounding factor is that the typical platform edge (presuming a low-floor LRV) is 10 to 13 inches [0.25 to 0.33 m] above the top of rail and street surface, appreciably more than the maximum curb height endorsed by the AASHTO "Green Book."

12.3.1 Alternative Configurations

Possible approaches to solving the problem of platform encroachment into traffic lanes include the following:

- Positioning the platform edge so that it is at, or very close to, the edge of the travel lane and biasing the track position in the traffic lane toward that edge. The drawback of this approach is that it places the static envelope of the rail car on the edge of the parking lane, where there could be a conflict with a badly parked vehicle, as noted above.

- As above, but increasing the width of the parking lane so as to prevent routine stationary vehicle conflicts. The drawback of this approach could be that motorists might erroneously presume that this extra space is part of the travel lane and collide with the platform anyway. Marking the outside edge of the parking/loading lane with an edge line might help mitigate this problem. Audible cues, such as rumble strip grooving of the pavement, might also assist in directing motorists back into the proper travel lane, although that detail might be disturbing to cyclists, particularly if the extra parking lane width becomes either an official or de facto bicycle lane.

- Transitioning the track from its normal position in the traffic lane toward the platform on the approach to the station and then back into the traffic lane on the departure end of a platform. A drawback of this technique is that it might introduce an LRT speed restriction at each station platform, regardless of whether there are any passengers to board or alight at that location. The transitional zones where the track is biased toward the platform would also become areas of possible conflict with parked vehicles as described above. Since it is typically necessary to keep the track tangent for a full LRV carbody module length ahead and beyond the platform, the length of the parking zone affected could be substantial and create problems in dense urban neighborhoods where mixed traffic operation is more likely and parking is usually at a premium. Arguably, the lateral movement of the rail vehicle might also confuse a motorist in a parallel traffic lane although this should be no more of a factor than a similar lane shift by a large truck or bus.

- Constructing a gauntlet track at each platform, so only those LRVs that need to handle passengers at that location need shift toward the platform. The problem with this approach is that it introduces significant additional complexity and cost into the track structure as well as additional steel into the pavement surface. It also presupposes that the LRV operator is diligent and can visually confirm the presence of a potential passenger on the platform sufficiently far in advance to throw the switch and make the lateral movement—something that seems rather unlikely. A "passenger stop request" button at the platform, similar to an elevator request button, could provide the LRV operator with advance notice, but that would add further complexity and cost. It might

also be subject to misuse. Another issue with a passenger stop request button is where it should be placed so it can be readily found by the visually impaired.

- Allowing the platform encroachment into the traffic lane, but providing signage, pavement markings, rumble strips, and other cues telling the motorists that they need to shift toward the other side of the lane. The problem with this is that the path of steerable vehicles is, ultimately, at the discretion of their drivers. Delineating an irregular and unnatural path, such as a reverse curve, within an otherwise straight roadway lane does not guarantee that every motorist will steer that path. (On the other hand, if a reverse curve is designed into a track alignment, as discussed above, it can be guaranteed that every rail car will follow it.) Accordingly, permitting any platform encroachment into the general traffic lane is discouraged.

- In combination with any of the above options, designers might consider placement of either active or passive warning devices on the leading edge of the platform. This was common on legacy streetcar lines, where both the tracks and a "safety island" loading platform might be one or two lanes away from the curbline. These warnings would typically consist of yellow blinking lights and illuminated signage directing motorists to the right or left of the platform. Such measures were not 100% effective at preventing motorists from colliding with the island, and a review of city newspaper articles from the trolley era will find more than a few photographs of the failures.

12.3.2 Motorist Perceptions

One issue that compounds the platform interface problem is that motorists tend to associate any longitudinal line in the street as a visual cue for where the traffic lanes are located. In the case of lanes which include streetcar tracks, not only the rails, but also any pavement joints paralleling those rails may be interpreted by motorists as indications of where their vehicles should be positioned. Differences in roadway surface color or texture (such as between asphalt and concrete pavement or between concrete and brick pavement) will also often be interpreted as the location of the edge of a lane even when formal pavement markings indicate otherwise. On a dark rainy night, the shiny rails may well be the most prominent visual cue, and a motorist who is instinctively following their alignment might be unintentionally directed too close to the platform edge. Since there is nothing the track designer can do to make the rails invisible, it is highly desirable that pavement joints and other informal lane delineations in the vicinity of the shared LRT lane do not conflict with the formal pavement markings. This will often entail extending the track area paving material laterally beyond what is required for structural purposes. See Article 12.8.1 for additional discussion on this point.

12.3.3 Tracks Adjacent to Median Platforms

If the street has a median that is also used for the station platforms, the problem is much easier to resolve. There is no abutting parking lane, and, the rail car, with its precise guidance, can run with its left side adjacent to an open median without danger of striking it. However, if elsewhere along the line the median area is used for a left-turn lane for general traffic, careful consideration must be given to potential clearance conflicts between the rail car, with its trajectory offset to the left very close to the edge of the through lane, and any vehicle in the left-turn lane.

12.3.4 Conclusions

The appropriate relationship of station platforms, lanes, and track location/alignment will not be the same on every project. Different solutions might be more appropriate for heavily trafficked boulevards versus quiet side streets, although there is something to be said for a uniform approach within any particular city. Track designers must work closely with the project's traffic engineers and station architects to determine the optimal solution for their project. Very often, the municipal traffic engineer will have veto power over this portion of the design, so it is advisable to include that agency in the discussions at an early date.

12.4 MINIMUM CURVE RADIUS

A likely location for an in-street LRT operation is a built-up area such as a central business district (CBD) where the street geometry is well established and lined with buildings and other substantial structures. In zones of this type, there is usually very limited opportunity to alter street widths or increase the radius of corner curb lines to accommodate the track alignment design. The tracks therefore need to be fit into the available space.

12.4.1 Light Rail Vehicle Limitations

In accomplishing this task, a major factor that comes into play is minimum track curve radius. It is possible to design tracks with relatively short radii. Curves with a 35-foot [10.67-meter] centerline radius were once common on streetcar networks, and some still exist. However, extremely sharp curves are generally undesirable for various reasons as cited in Chapter 4. In addition, the universe of available rolling stock becomes larger as the minimum curve radius increases. A common minimum turning radius for a majority, but not all, of current LRV models is 82 feet [25 meters]. Track curves of that radius are compatible with roadway intersection geometry where streets are wide, such as in the case of many European boulevards, but few business districts in the United States routinely have streets of sufficient width.

Use of 82-foot [25-meter] radius curves in situations where the streets are not of generous width has been done, but it typically results in a rail car trajectory that is not very compatible with either normal traffic patterns or efficient rail operations and is not recommended unless the rail line can be situated in exclusive lanes, such as in a transit mall, where ordinary traffic need not be a design factor. If that is not possible, it might be necessary to either change the routing to avoid making turns at constrained intersections or to procure light rail vehicles that can negotiate tighter turns.

12.4.2 Vehicles for Small Radius Turns

Several legacy streetcar systems still utilize very constrained track alignments that were fitted into existing street geometry a century or more ago. When it became necessary to procure new rolling stock for those routes, because the track geometry could not be redesigned to accommodate the minimum turning radius of many contemporary LRV models, the new cars had to be designed to accommodate the track geometry. Toronto has a well-established network of streetcar trackage with hundreds of curves with radii well below 82 feet [25 meters], dozens of which have a radius of about 36 feet [11 meters]. In 2008, when rail car manufacturers were

invited to propose on an order for a 204-vehicle fleet of new, 100% low-floor streetcars, designs compatible with the existing track geometry were forthcoming. Less recently, but still within the modern light rail era, a fleet of 112 rigid-body, high-floor LRVs was manufactured for operation on Philadelphia's streetcar lines, where the minimum curve centerline radius is 35.59 feet [10.85 meters]. The new car fleet was designed to accommodate these parameters. The legacy light rail systems in San Francisco and Boston have similar curve radius constraints. See Chapter 2, Article 2.4.1 for additional discussion on the topic of minimum vehicle curving limitations.

12.4.3 Overhead Contact Wire Considerations

The overhead contact wire must be kept within a relatively narrow width above the rails so that the vehicle current collector can properly follow it. When the track is in a sharp curve, it is often necessary for the OCS to include supplemental "pull-offs" between the vertical support points so as to keep the wires over the track, In extremely sharp curves of appreciable length, additional poles sometimes must be added. These extra pull-off wires and poles can add to the visual impact of the OCS, which may be at odds with the project's urban design goals. Hence, the introduction of additional curves in the track to facilitate traffic engineering goals may indirectly be at odds with other project objectives. The track designer may be in a central position to mediate such issues, even though the topic has relatively little to do with the track.

12.5 TURNING MOVEMENTS

12.5.1 Preferred Configuration

At any given location, the track alignment should mimic the path that a steerable vehicle would follow in the same circumstances. For example, when a rail car executes a right turn, that maneuver should begin in the rightmost travel lane, and when a rail car executes a left turn, it should begin either in the leftmost lane or from an exclusive left-turn lane, if there is one. Turning movements must not "cut across" adjacent lanes carrying through traffic unless the conflicting movements can be time-separated by signalization.

12.5.2 LRT-Only Traffic Signals

It is important to understand that unless the rail car is originating its turn from an exclusive lane, temporal separation cannot be achieved by means of special traffic signal phases. If the LRV is in a mixed traffic lane, even if it is given a signal indication distinctly different from that of standard highway traffic signals, the necessary physical separation of the different movements would not exist. A motor vehicle traveling in a shared lane but stopped by a red signal would physically block the movement of any rail car behind it, even during the phase of the signal cycle when a special "proceed" signal is displayed for the rail car. Similarly, a rail car halted by its "stop" signal would block the path of a motor vehicle behind it, even if that vehicle had a green traffic signal. An additional concern is that inattentive motorists in either the same lane as the streetcar or an adjoining lane, upon seeing the rail car begin its movement, may well presume that they may also proceed even if a conventional aspect traffic signal clearly indicates otherwise. This could create conflicts with not only the rail vehicle's path but perhaps also with any other vehicular traffic that is simultaneously executing a movement that is in accord with the traffic signals.

12.6 CLEARANCE ENVELOPE AND SWEPT PATH IN CURVES

As explained in Chapters 2 and 3, on a curved alignment the dynamic envelope of a rail car is widened. The outside corners at the ends of the body move farther from the track centerline on the outside of the curve while the center of the body (or each carbody segment, if there are more than one) moves farther from the centerline on the inside of the curve.

12.6.1 Difference from Rubber-Tired Traffic

For vehicles of any given length, the dynamic envelope of a rail car sweeps over a narrower overall area than does a steerable vehicle of similar length. This is because, unlike a truck, bus, or automobile, the rear wheels of a rail car follow exactly the same path as the front wheels. This minimizes the width of the swept area. However, this characteristic also has a negative consequence, which is discussed below.

12.6.2 LRV Tail Swing

When a rail car enters a curve from tangent track, the portion of the car body behind the rear axle will swing away from the track centerline in the direction opposite from that of the turn. If the rail car turns toward the right, the left rear corner will swing toward the left.

It is imperative that this “tail swing” does not encroach into an adjacent travel lane. This usually can be avoided by designing a long spiral at the entrance to the curve, possibly including a short section of skewed tangent alignment, so that the swing does not reach its full extent until the rear axle has moved laterally a sufficient distance in the direction of the turn to keep the swing within the marked traffic lane. In certain situations this might require some adjustment of lane widths, which is another example of the need for the track and roadway designers to work in concert.

The potential negative impact of tail swing can also be mitigated by adopting a vehicle body design with tapered ends. This can significantly reduce the width of the dynamic envelope on turns. For this, the cooperation of the vehicle designers is obviously required.

As noted above, tail swing should never encroach into an adjacent travel lane. However, there could be situations in which a tail swing into a narrow curb loading or parking lane would be difficult to avoid. This would almost always occur at the end of the parking/loading lane, in the immediate vicinity of an intersection. This is a zone where stopping in the curb lane is normally prohibited for a distance from the cross street that is designated by ordinance or statute. In practice these “corner clearances” do not receive a high degree of respect from motorists, even where prominent signage and pavement markings are used. Some motorists rationalize that they will not be penalized for parking “just a little bit” into the zone, or stopping in it “briefly.” These individuals need to understand that, even in the absence of a traffic citation, their vehicle may be vulnerable to damage from the tail swing of a turning rail car.

The edge of the dynamic envelope, if it protrudes into the curb lane, should be marked with a line defining the area swept by the tail of the turning rail car. This might be supplemented by specialized

signage, such as that shown in **Figure 12.6.1**, to give motorists who are considering stopping in a curb lane a clear indication of the potential consequences of parking within this zone.

A secondary benefit of physically marking the edge of a dynamic envelope on curves is to provide the rail car operator with a clear indication when a stationary motor vehicle has encroached into the swept path of the LRV. (This is generally unnecessary for experienced streetcar operators, who usually develop a good eye for detecting whether or not they can get past a potential encroachment.) On a few LRT projects a different paving material has been used to delineate the edges of the trackway, usually as an artistic statement rather than a traffic engineering control. This practice can tend to deemphasize the standard pavement markings and should be avoided. This is discussed further in Article 12.8 of this chapter.

Figure 12.6.1 Custom signage for tight clearance zones

12.7 STREET DRAINAGE, CROSS SLOPES, AND SUPERELEVATION

Where tracks are embedded in roadway paving, attention must be given to drainage of storm water. In urban roadway design, this is usually addressed by using a pavement cross section that slopes away from the center of the street and placing gutters and inlets at the curb with connections to a storm drain system to collect the gutter water.

12.7.1 Flangeway Drains

When embedded rails are included in the roadway, some storm water will be captured in the flangeways and flow longitudinally along the rails rather than transversely into the gutters. Special provisions, such as track drains, may be needed to deal with this runoff. The frequency of these flangeway drains can vary depending on circumstances. In all cases, they should be located at the low points of sag vertical curves and immediately upstream of any embedded special trackwork. LRT lines in frostbelt climates must carefully consider the probability of water freezing in the flangeways on streets with flat grades, thereby risking a derailment.

12.7.2 Roadway Crown and Track Cross Slope

The roadway crown, which is an essential element of the storm water drainage system of any street or roadway, often means that any tracks embedded in a mixed traffic lane will have something other than zero cross slope between the rails. Typically, the roadway is crowned on a 2% cross slope, resulting in about 1 $^{1}/_{8}$ inch [29 mm] of cross slope in the track. In tangent track, up to about 1 ½ inches (38 mm) of cross slope can be allowed without making the passengers feel uncomfortable. This includes both tracks in curves and those on a tangent alignment. Where the alignment is curved, the roadway design (per the AASHTO Green Book or other adopted standard) will typically dictate the superelevation of the tracks, not the formulae used for tracks in exclusive right-of-way. Sometimes this means that the value of Eu may be higher than the desirable maximum.

12.7.2.1 Codes and Jurisdictional Issues

If the street maintains a normal crown through both tangents and curves, there will be situations in which the outside track of a double-track line will actually have unavoidable reverse superelevation, exacerbating the natural effect of centrifugal force rather than compensating for it. While this negative superelevation may at first glance appear to be at odds with 49CFR213, paragraph 213.57, it is consistent with paragraph 213.63.

Moreover, embedded track in the street used only by light rail vehicles is not currently under the jurisdiction of the U.S. Federal Railway Administration. Hence, deliberate inclusion of negative cross slope in a track used only by LRVs is not a violation of 49CFR213 or any other code. In particular, it should be noted that since embedded track is a rigid trackform, as-constructed cross slope is extremely unlikely to ever change.

The same cannot be said about ballasted track since once a negative cross slope condition develops in ballasted track, it is highly likely to get worse. It was that prospect that led to the adoption of paragraph 213.57 in the FRA Track Safety Standards. It should also be noted that the impact of a slight negative cross slope on passengers is no different than it would be with transit buses operating in the same lane as the track.

In curved track, any adverse cross slope on the outer track of a curve must be factored in as a reduction in the allowable Eu for purposes of determining the allowable speed on the curve. However, it must be included in determination of the minimum spiral length.

12.7.2.2 Streets with Parabolic Crowns

The aforementioned cross slope usually does not exceed an acceptable gradient of 2% unless the street crown is a parabolic curve. Although that design is no longer common, where it exists, the cross slope in the inner lanes of the street could be very close to flat while those in the curb lanes could be considerably steeper than 2%.

If the track is proposed to be in the curb lane of a street with a parabolic crown, the steeper cross slope at that location may be more than can be acceptable, thereby requiring some reconstruction of that part of the street, with associated changes in drainage patterns, etc. However, if the existing curb heights are substandard because of decades of street resurfacing, the LRT project may be forced into substantial reconstruction of not only the lane proposed to be

occupied, but also the adjacent curb and sidewalk on one side and the through traffic lane on the opposite side. This can become particularly problematic in urban areas where buildings and doorway thresholds directly abut the rear edge of the sidewalk, making it difficult or perhaps even impossible to raise the grade of the sidewalk.

12.7.2.3 Special Trackwork Cross Slope

Even when the straight and curved track conforms to the normal crown of the street, it may be necessary to have a zero cross slope in segments where special trackwork is installed. In such cases, the track will need to warp from the ordinary cross level condition to zero cross slope over some distance. That distance will be dictated by the allowable twist for the design LRV.

12.8 PAVEMENT DESIGN FOR IN-STREET TRACKAGE—SEAM LOCATIONS

For a variety of reasons, concrete is commonly used as a paving material for embedded track in lanes shared with general traffic and at crossings. The use of concrete paving in the track area of a street otherwise paved with another material can have traffic impacts.

Construction of embedded track is expensive and disruptive to the community. Because of those factors, it is not unusual for projects to limit the width of the track slab to the maximum degree physically possible. On some streetcar projects, the track slab has been less than 8 feet [2.4 meters] wide—far less than that of the traffic lane containing the track. However, there are several good reasons why track slabs should be appreciably wider than the absolute minimum necessary to construct the track.

The seam of the two paving materials of disparate appearance (e.g., dark asphalt adjacent to a light gray concrete) is very likely to be wrongly interpreted by some motorists as an intended traffic marking such as a lane line, edge line, or stop line. This possibility should be taken into account when designing the details of the trackway paving.

If a track is located in a travel lane next to a curb lane used for parking or loading, the seam could be interpreted as a clearance edge line. In this situation, a true edge line would serve two purposes. One would be to define the edge of the travel lane for the moving traffic. The other would be to define the outer limit of the curb lane within which vehicles may be safely parked or stopped for loading.

If, for construction reasons, the minimum width of the track structure is such that it would produce a seam that would be inside the dynamic envelope of the rail cars, the resulting appearance could provide false assurance to motorists stopping or parking in the curb lane that their vehicle will be clear of the path of the LRV provided it does not span the seam. To address this, even where not required for structural reasons, the concrete paving used in the track lane should, at a minimum, extend for 6 to 9 inches [0.15 to 0.23 meters] beyond the outside edge of the dynamic envelope and preferably all of the way to the edge of the actual travel lane.

Similar problems could arise between two travel lanes. If the track slab is narrower than the lane in which it is located, the motorist could be presented with up to six longitudinal visual cues as to where the lane is located:

- The two edges of the actual lane, presumably delineated by pavement markings such as traffic paint.
- The two edges of the track slab.
- The two rails.

Such visual clutter presents the motorist with too many choices about where to position his/her vehicle—66% of which are misleading. To simplify things, it is highly desirable that the edges of the track slab match the location of the standard traffic markings separating the shared lane from the adjacent lanes. If concrete paving extends beyond the area between the two rails it should occupy the entire lane width. This reduces the visual cues concerning lane location by 33%, making it more likely that motorists will follow a predictable path.

Ideally, the paving material in a lane shared by rail cars and general vehicular traffic should be the same as that used in the adjacent lanes. This allows the standard markings to be the predominant guidance for motorists.

12.9 SPECIAL TRACKWORK IN STREETS

Ideally, switches should not be located in a mixed traffic lane. There are several reasons for this, including some having to do with costs. Perhaps the most important reason is that a switch in a mixed traffic lane needs to be inspected and maintained on a regular basis. Those activities expose the maintenance employees not only to the hazards associated with working along any active railway track but also to the dangers inherent in working in a public street. For that reason alone, the track alignment should seek to locate switches in some form of exclusive right-of-way. That is, however, not always possible. When such is the case, the track switches for rail car operations in trafficked lanes must be compatible with the roadway environment. Two elements need to be considered.

12.9.1 Switch Hardware

The switch itself must be one that is designed for use in a paved street, not a railroad switch adapted for an unintended use. Flexive tongue switches of European design that are compatible with street environments are commercially available from multiple sources, including some North American manufacturers. These designs take into account that the points will be driven over by rubber-tired vehicles. The gaps between the point and the rail, which are inherent and unavoidable elements of every track switch, are kept to a minimum so that the tire widths of motor vehicles are sufficient to span them.

12.9.2 Switch Location

Moreover, the primary challenge for the track designer is not the selection of the proper hardware, but how these switches are deployed in a street setting. Both vehicular and pedestrian movements need to be considered, and one can affect the other.

12.9.2.1 Hazard Issues

Track switches of any type, even those designed for use in trafficked roadways, are incompatible with areas where people walk. The gaps between the movable points and other rails are

potential hazards. The risk that the "hole" on the walking surface might cause pedestrians to trip is obvious. In addition, the housing of certain types of switches has a metal surface that is much wider than a single rail. Such surfaces can be slippery when wet.

However, there is an even more serious concern. Customarily, there is a power mechanism that moves the points from one position to the other, closing the gap on one side of each point and opening one on the other. If the foot of a person (or an animal, such as a guide dog) is in that gap when it the switch is being remotely thrown by the power switch mechanism, or even on top of the switch blade, severe injury would likely result. For this reason, switch points should never be located either in or close to a marked crosswalk or any other legitimate pedestrian path. In the event that a preliminary design indicates that the ideal location for the points of a switch would be in or close to a pedestrian crossing, one or the other should be repositioned.

12.9.2.2 Pedestrian Crosswalk Locations

There is not much flexibility in pedestrian crossing design. Best practice calls for designing a pedestrian crossing so it follows the path along which people would most naturally walk in the absence of a physical constraint. If a formal crosswalk is shifted too far from a natural walking route, a significant proportion of pedestrians will ignore it and instead take the natural path. Furthermore, crosswalk markings are not visible when covered with snow and can also be less noticeable during hours of darkness, especially if the road surface is wet and reflecting street lighting. For these reasons, it is best that pedestrians have no reason to ever encounter track switches and other special trackwork components, such as the frogs where two rails intersect.

12.9.2.3 Advance Switch Positions

In addressing this concern, there is more flexibility in track design. Should the ideal location for the switch point be in a crosswalk or other legitimate pedestrian path it might be possible to reposition it a few feet upstation or downstation simply by altering the curve radius within reasonable limits. An alternative might be to install the switch points a safe distance ahead of the crosswalk with the frog beyond it, connecting the two with a section of gauntlet track as shown in the photo presented as **Figure 12.9.1**. In that photo, a double tongue flexive switch has been installed nearly 130 feet [40 meters] in advance of the frog of the turnout. This position achieved three goals:

- The switch is not in a pedestrian crosswalk, which can be seen at the top of the photo.
- The switch is not in a section of curved track, which would require a custom fabrication.
- The switch is not in the middle of a street intersection, where inspection and maintenance would be hazardous.

Figure 12.9.1 Advance switch

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Chapter 13—LRT Track Construction

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CHAPTER 13—LRT TRACK CONSTRUCTION

13.1 INTRODUCTION

The purpose of this chapter is to offer discussion on procedures and topics that are relevant not only to the construction of all types of trackforms but the entire construction process in general. Many instances of unsatisfactory construction occurred in part because the designers had little or no field experience. The major focus will therefore be on the designer and particular aspects of the construction process that should be understood when designing track for a rail transportation system. This is not written as a detailed procedure for construction but rather as a primer to explain what the designer needs to understand about the construction process since details of the design can have significant effects on the cost and/or the integrity of the system. Some approaches are difficult to build and some are simple, yet they both may function in an identical manner. Track systems that are more difficult to build can also cause more problems for persons maintaining and operating the system. Designing a very complicated track system may cause premature failure since it may also be nearly impossible to build accurately.

13.2 GENERAL REQUIREMENTS—ALL TRACKFORMS

13.2.1 Project Procurement Methods

There are many methods by which an owner can procure a rail transit project. This article explores the characteristics, advantages, and disadvantages of each of the procurement methods listed below. In each case, the obligations of the designer will vary somewhat. In the case of alternative project procurement methods, the track designer's obligations may vary depending on whether he or she is employed/engaged by the project's owner versus the constructor. Each method has a different "degree of difficulty" and different pitfalls from the perspectives of the owner, the designer, and the constructor.

13.2.1.1 Design/Bid/Build

The traditional approach to producing a transit system will utilize this method. An owner, typically a transit agency but sometimes a municipality, has a vision of a rail transit line connecting two or more points. The owner hires a designer to develop that vision into 100%-complete, bid-ready plans, specifications, and cost estimates. The completed plans and specifications are publically advertised for bids, and a construction contractor or contractors are selected to actually build the project.

The distinct advantage to this method is the controls it provides to ensure quality in the finished product. If the design is 100% complete prior to soliciting bids for construction, the desired finished product is generally well defined in the bid documents. The interface issues have typically been thought through, since the entire project will have gone through a comprehensive design review process.

Once the design work is complete and signed off on by the owner or the end user, the bid package is developed and offered to the private construction sector to prepare a cost estimate. The bids are received, and the lowest responsive and responsible bidder is chosen to build the job. When the construction is complete, there must be a written sign-off that the project was built

in accordance with the plans, specifications, and other contract documents. "As-built" drawings are produced, and final payment is made to the constructor(s).

The next step is to turn the system over to the operator, which may or may not be the same entity as the owner. The operator will test the subsystems of the transit line and then operate test trains, a process that is collectively known as systems integration testing. This process must pass certain guidelines for performance and generally must achieve a 3-month period without incident before the system can be deemed ready for revenue service.

During this period, a Maintainer will be selected. Usually, but not always, this will be part of the operator's organization. The Maintainer performs periodic inspection and maintenance activities so that the system remains within specified minimum standards applicable to and adopted by the project. Depending on jurisdictional issues, maintenance criteria may incorporate regulations and guidelines of the Federal Railroad Administration (FRA) or the American Public Transportation Association (APTA). (Note: As of 2010, the Federal Transit Administration (FTA) does not regulate maintenance of transit infrastructure and systems but instead defers such authority to the states.) No matter which entity oversees operation and maintenance, it is imperative that minimum maintenance standards are developed for the safe operation of trains. See Chapter 14 for detailed discussion of the maintenance of LRT track and trackways.

There are some distinct advantages and disadvantages to the traditional design-bid-build approach.

- On the plus side, the system is generally built exactly to match the owner's requirements and the designer's specifications provided checks and balances are in place to ensure this occurs. Another advantage is that pricing will be more competitive during the construction phase.

- The major disadvantage is the length of time it takes to get from the initial planning concept to completion of the project and the commencement of revenue LRT service. It is not unusual for this to take 10 to 15 years or more. The process is linear, and therefore each step in the process must be 100% complete before moving on to the next step. As the schedule stretches, the overall cost for the full completion of the project, measured in terms of dollars at the time of their expenditure, can escalate dramatically. However, it should be noted that the majority of that schedule is typically consumed not by final design engineering and construction but rather by the planning and environmental clearance activities. Those efforts are still required even if an alternative project delivery method is utilized for final design and construction.

13.2.1.2 Construction Management-General Constructor (CMGC)

This process, sometimes also known as Construction Manager at Risk, typically requires enabling legislation, since the construction team selected isn't necessarily the low bidder. The owner will first hire an engineering firm to design the project and subsequently provide limited oversight of construction. While the design is underway, but well before it is completed, the owner will pre-qualify construction teams and invite selected teams to respond to a formal request for proposals (RFP). The teams are often composed of a management firm and one or more construction firms. The prequalification and proposal process may be very extensive and includes not only

preliminary pricing of the in-progress design, but also the qualifications of the candidate CMGC firms and their proposed technical approach to the construction of the project. The owner, usually aided by an engineering firm, will review all proposals submitted and select the CMGC team that they believe will provide the best value.

Once the CMGC firm is chosen, they will collaborate with the designer, providing input to the design while they concurrently refine the construction cost estimate. As constructible portions of the design are completed, the CMGC firm negotiates an agreement with the owner as to the final price for that part and then goes to work. This includes both building the job and overseeing the construction activities of subcontractors so as to ensure that it is built to the owner's and designer's specifications. The CMGC firm is also tasked with solving problems that may occur on the project. They are generally given very tight schedules to produce the project. Liquidated damages can be high, introducing an element of risk. Once the job is complete, it is then turned over to the owner to operate and maintain.

CMGC should be considered when construction contractor input during design is deemed critical and when significant construction-related impacts to the public are expected and a proactive process is needed for their mitigation. An advantage of CMGC is that the owner has the opportunity to develop the RFP with criteria that focus on meeting the objectives for a successful project. This will reduce the risk of awarding the contract to a constructor who lacks the experience and capacity to perform the work. Another advantage is that the owner does not need a large staff to administer the project. This may save the owner both money and liability. A disadvantage is that now the CMGC, in the legitimate pursuit of a profit, might, in the absence of third-party oversight, take shortcuts or overlook deficiencies in the construction. This problem can be minimized if diligent prequalification is performed so that a high-quality project can be achieved. If the selection scoring process is heavily weighted toward getting the lowest first cost, the completed project could possibly have maintenance costs appreciably higher than expected.

13.2.1.3 Design/Build

This innovative approach to project procurement has become very popular. Like CMGC, it does not select based solely on a low bid, and enabling legislation is usually required for any public agency to use this method. The owner will generally produce some drawings showing the system in basic format, leaving the details to the design/builder. The drawings are typically taken to a 30%-completion stage although the package may include standard details that are at or near 100%, particularly if the owner is an existing LRT operator. The owner will then issue a request for proposals (RFP) to the contracting community. Teams will form consisting of a designer and a construction contractor. Typically the constructor will be the lead entity. Similar to CMGC, there is generally a prequalification process, and certain teams will be selected as qualified to perform the work. Those consortiums will then be requested to offer pricing on the project. The owner will identify a grading system incorporating components for both price and each team's technical approach to the design and construction. The scoring system must be identified during the RFP stage.

The next step is to receive proposals and begin an evaluation phase to choose the entity with the best score. The evaluation process may take several months to complete before a design/builder is identified.

As with CMGC, the schedules can be very short since construction can begin before the entire project is designed. This is an advantage but could also be the cause for modifying the work or the design if the pace of construction exceeds the ability of the designer to fully investigate all of the interfaces between facilities and systems.

Once the project is built and signed off on, it is then turned over to the owner for testing and operations. Issues concerning the system operator and maintainer are as previously discussed under the traditional approach.

The advantage to this procurement method is that the owner will get the system much sooner than the methods discussed above. A significant disadvantage is that unless the 30% design, particularly the design criteria and the specifications, are very specific as to the quality of materials required, the system may be built with materials that meet only the minimum requirements. Since the design/builder will not have any long-term commitment to the system, there is little incentive to provide high-quality products.

13.2.1.4 Design/Build/Operate/Maintain (DBOM)

For the owner that simply wants to have a project, but does not want to be involved in day-to-day details of operation and maintenance, the DBOM process is a good method to have a system operational in a very quick manner. The first stages of the project are functionally similar to Design/Build but with the addition of operations and maintenance criteria with which the Concessionaire must comply for some period of time. The concession period could extend for several decades. This DBOM team may also have responsibilities for other aspects of construction such as community relations, mitigation of unforeseen conditions, and public artwork, just to mention a few. These types of responsibilities will add risk that the Concessionaire must anticipate in the bid price and manage during the term of the contract. Because of the risk management issues associated with DBOM, the Concessionaire will very often be a joint venture with a financial services firm as a key partner. Prequalification of DBOM teams is an extremely important part of the project procurement process since failure of the Concessionaire prior to completion of the contract term could have severe impacts.

The biggest advantage of the DBOM procurement method is that the Concessionaire has an incentive to use top quality products during construction since it has to maintain what it has built. Accountability is an important part of the process. As with Design/Build, the construction can begin before design is complete. In many cases, testing can commence before construction is complete, and segments of the route may open for revenue service before the entire job is completed. DBOM project procurement can be non-linear, and the implementation schedule can be stacked in many different ways from initiation of design through to the commencement of revenue operations.

13.2.2 Track Material Procurement

Procuring trackwork materials in a timely manner for the right price is very important to the project and will set the tone for construction. Many of the materials used in LRT track construction are “long lead items,” and their procurement can range from 6 to 12 months and even more. Delays can occur if the material is not onsite and approved for installation prior to construction. Determining who will purchase the material is an important decision and one that can have large

ramifications for the success of the project, particularly with respect to the schedule. The two major material procurement options are owner furnished and constructor furnished, and each has advantages and disadvantages.

13.2.2.1 Owner Furnished

13.2.2.1.1 Description of Process

If the owner furnishes materials to the constructor, there will be a separate procurement contract or contracts that are prepared and awarded prior to the construction contracts. This overlaps the fabrication time for the materials with the time necessary to complete the design, bidding, and award of the construction contracts.

Ideally, this process begins with extremely detailed specifications and drawings. In addition to the normal reviews by the designer's team and by the owner's capital projects staff, these documents should be reviewed by the selected maintenance organization since they will ultimately inherit the system. Many problems can be avoided by a simple review process. If this is the first procurement package for a new system, the materials procured may become de facto standards for the owner and any LRT system extensions or renovations may need to be compatible.

The next step is to advertise for proposals from track material vendors. Several national industry publications are available for this, and advertisements should not be limited to the local newspaper. However, note that the lead time for some print publications can be lengthy, and cutoff dates may be several weeks in advance of actual publication. Nevertheless, they can be a good way of providing candidate vendors with advance notifications of the bidding and advising them to monitor the owner's website for detailed information as to when the bid documents are actually available.

If allowed under the state regulations under which the owner operates, it may be desirable to pre-qualify vendors on large purchases. A pre-bid meeting may be appropriate but not always necessary. Particularly in the case of complex special trackwork, where the bidder may need to do some additional design work to adapt the project design to standard components, an ample amount of time should be given for bid preparation as well as time allotted for questions.

The delivery schedule is an important part of the contract documents and should reflect liquidated damages since the constructor may file a claim if material delivery is delayed past a promised date. Bonds should be required just in case the vendor does not perform, and these bonds should be of a value to cover delays, expected revenue, claims, litigation etc. Bear in mind that the more risk taken on by a contractor, including materials vendors, the higher the bids will be. If there is a possibility of risk sharing, that option should be explored. Naturally, all such bid documents should be closely reviewed by the owner's legal counsel. In general, any particular type of risk should be borne by the party who is in the best position to manage the risk and implement mitigation measures. Some types of risk can best be managed by the owner while other risk issues are completely outside of the owner's control. On a complex project with multiple prime contractors, the owner is very often in the best position to manage overall risk.

Once the lowest responsive and responsible bidder is identified, the procurement contract can be awarded. Execution of work must closely adhere to the contract requirements. A quality

management process that includes both quality controls by the vendor and monitoring by the owner and the designer must be followed, including documentation, so that the products furnished to the constructors meet both the specified requirements and the schedule.

13.2.2.1.2 Advantages and Disadvantages to the Owner

Advantages of owner furnished to the owner:

- On a project that is an extension of an existing LRT system with previously established standards, if the owner furnishes the material to the constructor(s), they can be confident that the new material will be compatible with their existing system.

- If it is new construction, material procurement can take place prior to the RFP for construction, compressing the overall schedule. If this method is done properly, the possibility of project delays due to materials being unavailable is greatly minimized.

Disadvantages of owner furnished to the owner:

- If the owner is not diligent with the procurement and the constructor runs out of material, there will probably be some claims involved for lost time, lost continuity of work, loss of anticipated profit, and extended overhead.

- The owner must have the expertise to manage this activity, either with in-house staff or through a construction manager who can act as an extension of staff. The owner or the owner's designee also must closely monitor the issuance of materials to the constructors and verify that they are not being careless with or wasting the items. Rail can be particularly problematic if the constructor does not carefully plan cuts so as to avoid generating a large pile of unusable short lengths. It is recommended that the bid documents for the construction very specifically itemize by type and quantity any owner-furnished materials that will be issued along with a stipulation that the constructor is responsible for making up any shortfalls.

- The owner takes the responsibility and risk of ensuring the quality of material furnished to the constructors. Typically it is necessary to add personnel (either in-house or consultants) to inspect each delivery to ensure conformance with the specifications.

- The owner must have secure staging areas to stockpile the material and protect it from damage and theft.

13.2.2.2 Constructor Furnished

13.2.2.2.1 Description of Process

The constructor takes the responsibility of furnishing the long lead track materials just as it would handle more routine construction materials. The same sorts of drawings and specifications still need to be developed; however, the risk is now in the hands of the constructor and its suppliers. If the owner decides to intervene for any reason at this phase, costs will likely escalate and claims will likely ensue from both the constructor and its supplier.

The owner must have a process in place to review and approve shop drawings prior to the constructor purchasing the material. Bear in mind that if any changes are identified during those shop drawing reviews, they may result in a claim or change order that in most cases will add cost to the project.

In design/build projects, the owner may not have any direct control over the shop drawing process. Nevertheless, the owner will virtually always maintain the right to review the contractor's design for its compliance with the performance specification that was the basis for the design/builder's proposal. The owner will want to be certain that all materials conform to not only the specification, but also the quality program that the constructor would have submitted, either with the bid package or shortly after award of contract.

13.2.2.2.2 Advantages and Disadvantages to the Owner

Advantages of constructor furnished to the owner:

- The constructor now accepts the risk of inadequate or defective material and can be held accountable.
- The constructor is responsible for interface compatibility and for quantities.
- The owner may get better pricing since the constructor can cover some risk in the procurement of material and may have better buying potential.
- The owner will not need as many field personnel or as much equipment during the receiving stage.
- The constructor now accepts the risk of transportation of the material from the supplier, including unloading in its own staging areas. The owner does not have the responsibility to protect the material from damage or theft.

Disadvantages of constructor furnished to the owner:

- The construction schedule must now include time for procurement of long lead items. In a traditional Design/Bid/Build project procurement process, this can be a distinct problem. It is generally not a problem under alternative project procurement methods such as Design/Build and DBOM.
- If the owner is an existing transit system with an established inventory of spare parts, procurement of the materials through the constructor increases the probability of getting materials incompatible with the existing system since one level of controls has been eliminated. This could dramatically affect both the Maintainer's required spare parts inventory and the service life of the system.

13.2.3 Design Concept

Both the clarity of the design and the constructability of the design are absolutely crucial to the success of any project. The intent of this article is to address those topics, offer some thoughts concerning pros and cons of various design concepts and offer counsel concerning the degree of

construction difficulty. Some designs are easy to build, some are hard to build, and some are impossible to build. Finding the right mix is the challenge in producing a safe and reliable system.

13.2.3.1 Clarity of Drawings and Specifications

Under the conventional Design/Bid/Build project procurement method, the clarity of the 100% bid-ready design documents is imperative. Prior to advertisement they must be scrutinized to make certain that there are no ambiguities concerning the designer's intent on critical items that might invalidate assumptions concerning the operation and maintenance of the system. It is the nature of all construction to build projects at the least cost and the highest possible profit to the constructor. If there are flaws or irregularities in the bid documents that must be corrected after contract award so that the owner gets the project it wants, claims, change orders, and possibly even litigation will ensue.

Trackwork is deceptively simple in appearance but very complex in its details. The design engineer preparing the drawings and writing the specifications should be qualified and very familiar with the entire process from inception to completion. Many rail transit projects have stumbled because the trackwork design was assigned to a junior-level civil engineer with inadequate supervision and oversight. Particularly under those circumstances, using a second, third, or fourth pair of eyes to review bid documents before they are advertised can save millions of dollars in the overall project as well as embarrassment to the designer and the owner. Extreme care must be taken when using design criteria, drawing details, and specifications from other, and nominally similar projects. In many cases, there are appreciable differences in the functional requirements of the projects. The borrowed information may also be outdated. The rail industry is constantly evolving, and new processes and better materials are frequently available. On the other hand, new products are often developed by suppliers who themselves are new to the industry and therefore don't understand the service requirements that are imposed on railway infrastructure. Therefore, careful research and testing are imperative.

13.2.3.2 Keeping the Maintainers in Mind

When designing and constructing a rail transit system, it is highly advisable to include the people that will be maintaining the system in any decision-making process. This is especially important on any mature system that already has a maintenance organization. Asking for their advice or review of documents will help avoid compatibility problems as well as facilitate the development of a unified team that will help the project succeed. In the case of a starter system, the Maintainers are often not identified until the construction is well underway, by which time there is little flexibility for design detail changes. In those cases, it may be prudent to include some personnel with rail transit maintenance expertise during the design reviews. Their expertise can often make the difference between a successful project and one that needs extensive maintenance and possibly even reconstruction in only a few years.

13.2.3.3 Traction Power Stray Current

Corrosion due to stray traction power current can be devastating to a track system and has caused failures and premature degradation on many transit systems. The decisions made at the beginning of the project with respect to control of stray current will dictate the success or failure for decades to come. There are numerous examples of the damage stray current can do to the system and how it can affect the safety of persons riding and maintaining the system. While

Chapter 8 covers this topic more extensively, the discussion below addresses stray current from a construction perspective.

In a typical electrified rail transit system, the traction power current leaves the substation using either an overhead catenary system or a power rail (also known as “contact rail” or “third rail”). In general, these systems are DC (direct current) at a potential of 600 to 750 volts. The current then passes through the propulsion control system and the traction motors. The return path from the motors to the substation is through the vehicle trucks and wheels to the running rails. The rails carry the current back to the negative bus in the substation.

There is no such thing as a perfect conductor that offers zero resistance to electric current, and the rails are no exception. Because of this resistance, a portion of the current will leak off the rail and seek other paths back to the traction power substation. This is known as “stray current.” The stray current seeks other grounded structures and follows those paths on a zig-zag path back to the substation. Wherever this current leaves one metallic conductor (such as the rail) and jumps to another conductor (such as a water line) corrosion is initiated on whichever conductor the current is leaving.

Ideally, the rails are sufficiently insulated that only trace amounts of stray current leak from them. However, there are no perfect insulators, particularly in the gritty environment typical of railway tracks. So, additional measures are usually necessary to protect surrounding structures, particularly any steel reinforcement in concrete structures supporting the trackway. As is described in more detail in Chapter 8, one method is to electrically bond all of the reinforcing steel together so that there is no difference in electrical potential between them. The other is to use epoxy-coated reinforcing steel so that current flow from one bar to another is prevented.

Regardless of the steps taken to control stray currents, it is highly recommend that a baseline survey of existing stray current be performed prior to any construction activities. Stray current can originate from many sources that have nothing to do with the rail transit line. Identifying the sources and intensities of any such background stray current is essential to understanding whether the rail isolation measures taken are effective. This information will help prevent disputes as well as protect the new system from damage due to outside sources.

13.2.3.3.1 Stray Current Isolation

The method of isolating the running rail will differ depending on whether the trackform is “open,” with the rails fully exposed, or “closed,” with only the running surface on the top of the rail head visible. Open trackforms, which include ballasted track and direct fixation track, are generally much easier to isolate since the electrical isolation can be confined to the rail fastening system. Closed trackforms, which include embedded track, grass track, and railway/highway at-grade crossings, are far more difficult to insulate and keep insulated. This is because not only does most of the surface of the rail need to be encapsulated, the isolation system needs to be able to survive and function within a generally dirty environment, particularly when in a public street. Further, in the case of embedded track and grade crossings, the insulated track needs to function under not only the loadings imposed by rail vehicles, but also the wheel loads of rubber-tired traffic, including heavy trucks. In addition, electrical isolation systems for closed trackforms are often abused by a wide variety of chemicals, particularly in cold climate zones where de-icing products are used to keep the streets clear.

Products that can be used for electrically isolating the track are discussed in Chapters 5 and 8, and that information will not be repeated here. Note that the electrical isolation system should not prevent the rail from deflecting under load. The rail should be able to vertically deflect about ⅛ to ¼ inch [3 to 6 mm]; otherwise, the system will very likely radiate noise and vibrations and corrugations will develop on the top surface of the rail. This resiliency is very important in embedded track because the vibration from stiff track can degrade the surrounding concrete in a short amount of time. See Chapters 4 and 9 for additional discussion of track stiffness.

Regardless of the trackform or the method of electrical isolation, the most important factor for success versus failure is the quality processes that are used during isolation material manufacturing and its installation during construction. Unless a comprehensive quality program is developed and properly executed, the electrical isolation systems will fail, stray currents will ensue, and damages will occur. Failed electrical isolation systems have resulted in major corrosion damage to underground utility lines. There have even been serious injuries to personnel due to leakage of the return current. The cost of correcting these damages can be huge compared to the cost of having done it right the first time.

Locations of stray current leakage can be nearly imperceptible under visual inspection. One pin-sized hole in any encapsulation method can cause a “hot spot” of leakage, and corrosion of the rail steel will commence. Once the current begins traveling along reinforcing steel, utilities, and other buried structures, the problem is compounded by additional corrosion each time the current leaves one unintended conductor and leaps to another.

13.2.3.3.2 Stray Current Collection

Building track that is electrically isolated can be easy compared to the task of maintaining those insulation systems. Both embedded track and open trackforms in tunnels often require continuous maintenance attention to keep them clean so that stray currents don’t simply bypass the insulation systems. Even with diligent housekeeping, which few publicly funded transit systems can afford, some trace amounts of stray current are inevitable.

Because of this inevitable leakage, some transit systems incorporate measures to safely collect it and carry it back to the substations. This requires that all surrounding steel, including reinforcing steel, be electrically bonded together into a continuous grid. Epoxy-coated reinforcing steel is typically not used in this case. This grid of steel is now connected to a grounding cable leading back to the substation. Extreme care must be taken since any discontinuities will create “hot spots” that could allow uncontrolled stray current leakage and even risk life-threatening injuries to personnel. Stray current collection is therefore generally used only as a last resort back-up to stray current isolation. See Chapter 8 for additional discussion on stray current and corrosion control.

13.2.3.3.3 Inspection of Electrical Isolation Construction

The importance of inspecting those measures taken to ensure that traction power return current does not leak from the track cannot be overstated. Particularly in embedded track, it is all too easy for a fault in the insulation system to be concealed by later construction. This includes damage that is caused by the subsequent work. The electrical isolation measures therefore must be inspected during each stage of construction so as provide assurance that they are functional.

If electrical isolation is only checked at the end of the construction, any requisite repairs will be far more expensive than if the issue was discovered earlier.

13.2.3.4 Tolerances

Realistic tolerances for manufacturing and construction must be identified in the bid documents and coordinated with each other so that the track system can meet the expected performance requirements. Manufacturing tolerances for track materials are typically well defined in industry documents such as the AREMA *Manual for Railway Engineering* and their *Portfolio of Trackwork Plans.* The typical construction tolerances for LRT track construction have been ⅛ inch [3 mm] on everything, including gauge, line, and surface. These tolerances, which are driven by ride quality issues, are much more restrictive than most railroad track construction contractors are used to dealing with, and it is important that they are clearly defined and enforced.

Some designers include even tighter tolerances in the bid documents, often rationalizing that by specifying an extremely tight dimension, there's a better chance that the constructor will achieve some looser figure that is the designer's real goal. This practice is not in the best interest of the owner. Establishing construction tolerances that are not readily achievable, such $^{1}/_{16}$ to $^{1}/_{32}$ inch [1 to 0.5 mm] on gauge or alignment, will only cause increased cost without actually improving quality. The materials being used include fabrication tolerances, and the accumulation of allowable fabrication tolerances within subassemblies plus their assembly can make ultra-tight construction tolerances virtually impossible to achieve. The bidders will protect themselves against the possibility that the owner might actually expect these ultra-tight tolerances by including a contingency in their bid prices. That way, if the owner agrees to waive a particular tolerance in return for a price credit, the constructor will not actually lose any money. Other constructors may elect to not bid the project rather than deal with the risk and aggravations involved in unreasonably tight tolerances. Specifying rational construction tolerances that can meet the actual quality requirements and be achieved with commonly available materials is in the best interest of all parties.

It is important to understand that construction tolerances have absolutely nothing to do with either maintenance limits or safety tolerances. Those factors are relevant only to the Maintainers of the system, not the constructors. The limits called out in the FRA and APTA Track Safety Standards indicate conditions at which either corrective repairs are required or train speeds must be reduced. They have no relationship to construction tolerances and cannot be used as a justification for not meeting the tolerances specified for newly constructed track.

13.2.3.5 Special Trackwork

Details of special trackwork components are discussed in Chapter 6. The discussion here relates to issues the designer needs to understand about how special trackwork is fabricated and installed.

As with all trackwork, the needs and issues of the Maintainer should be kept in mind during design. Simplification of the spare parts inventory is a high priority for any maintenance staff and is extremely important when it comes to special trackwork. Having different styles, shapes, and sizes of special trackwork can make maintenance activities much more complex and difficult than necessary. Non-interchangeable parts increase the risk that the Maintainers will undertake

emergency field modifications that could unintentionally jeopardize both the integrity of the track and the safety of the passengers.

13.2.3.5.1 Special Trackwork Fabrication Inspection

Inspecting material and processes during all phases of the project is important. It is therefore strongly recommended that special trackwork be fully assembled in the fabricator's shop for the final inspection. If the first time the whole layout is assembled isn't until after it arrives at the job site, it can be virtually guaranteed that some component will not fit within specified tolerances.

So as to eliminate any future claims by the constructor that the special trackwork is defective, the constructor should be required to participate in the shop inspection even if the procurement and installation of the special trackwork are in separate contracts. This co-inspection also gives the constructor the opportunity to coordinate with the fabricator on issues related to packaging and shipping the special trackwork so as to facilitate the constructor's requirements.

The owner's requirements for plant inspection of special trackwork should be clearly spelled out in the bid documents. Typically, the special trackwork is fabricated far away from the actual project site, and travel costs and logistics preclude casual inspection visits. It is therefore important that the material actually be ready for inspection on the stipulated date. The contract should stipulate that the manufacturer's own quality processes will have identified and corrected any deficiencies prior to the arrival of the owner's inspector and that the inspector be given sufficient time to perform thorough inspections without interference from other plant activities.

Ensure that the manufacturer is following a quality assurance plan specific to its means and methods as well as the requirements of the contract. If the manufacturer is ISO 9000 compliant, all records should be submitted for compliance. Sharing, cataloging, and controlling documents should be the norm. Requirements for storage of record documents should be established by the contract.

13.2.3.5.2 Shipping, Handling, and Installation

Basic requirements for packaging and shipping should be identified in the contract, and the actual handling methods proposed to be used should be submitted for acceptance. The method to be used for unloading trucks is important so that the manufacturer loads the trucks accordingly. Few jobsites are equipped with loading docks, and therefore must be unloaded from the side rather than the end. Packaging also must be suited for the particular equipment used to unload trucks. These types of practices should be identified and resolved before any shipment of material. This will reduce, if not eliminate, possible damage due to poor unloading practices. It is advised that all equipment be identified for the safe unloading of material. The manufacturer's handling instructions should identify recommended equipment and methods for unloading and installing the material. The constructor's quality group should follow up to verify that the procedures are being followed.

13.2.4 Preparatory Work

This article should be considered as a checklist for the designer. The objective is to highlight to the designer some stumbling block topics that may arise during the course of construction with the goal of identifying constructability issues at an early date when it is still possible to resolve

them at lower cost. If such matters are not discovered until the construction phase, it typically results in delays, design changes, change orders, claims, counterclaims, and possibly even litigation.

13.2.4.1 Site Plan

It is recommended that the designer develop an overall site plan of the project, including staging areas, material stock pile locations, and access opportunities and limitations. The site plan should identify access points, construction roads, and an overall flow of trucks for the delivery of material. Locations for office trailers and fuel storage should also be identified. This will enable the design team to more accurately understand how the project will be built and how much it will cost to do so. This site plan can be included in the bid documents as a "For Information Only" sheet so that bidders understand and can benefit from the designer's perspective on the project's construction.

Further development, submission, and approval of the final site plan by the constructor(s) should be a contract requirement before site mobilization begins. The constructor's version will be much more detailed and will show access points, staging areas, and flow of construction equipment. Staging areas should be shown, and areas identified for stockpiling of each type of material. This plan should show the entrance points for material deliveries and exit points for all vehicles. Some material is delivered by tractor and trailer, some is delivered by rail, and some by small box-type trucks. A well conceived site plan will help the constructor, designer, owner, and any third-party stakeholders (such as the community) reach a common understanding of the construction process and achieve a goal of "no surprises."

13.2.4.2 Accessibility

Access to the work site has a major effect on the smooth progression of any construction project and can make a big difference in the cost of construction as well as the schedule. Access points and staging areas need to be strategically located for continuity of work. This is particularly important on a linear and sequential project such as building track. For example, if on a ballasted track project, ballast can only be stockpiled at one end of the job, there could be disruption to other construction activities so as to allow ballast trains to pass through. There are also risks associated with double handling of ballast and its contamination, degradation, and segregation before it can be placed in the track.

13.2.4.3 Continuity of the Work

Any time an activity comes to a halt and must start again, two things happen:

- The follow-on activities also come to a halt and "stack up," which means that in order to establish continuity again, each one must start fresh and advance far enough to allow each of the other activities to begin.

- Another learning curve takes place, and this is where cost versus productivity is impacted. In some cases, the constructor may have moved on to other areas or even other projects and when the time comes to begin again, there may be new personnel involved, which causes low production and can affect quality if proper quality control measures are not implemented.

13.2.4.4 Vehicular Traffic Management

Especially if a project is in an urban environment, vehicular traffic must be taken into consideration. Detours should be identified and permits requested well in advance of construction. In embedded track areas, streets may need to be closed for track construction and intersections closed or rerouted. Sometimes it is necessary to close street blocks without also closing the adjacent intersections. This can add cost to the project as any discontinuity of constructing the track will add cost due to remobilization. Longer work zones can be constructed at less cost per unit length than short stretches.

13.2.4.5 Pedestrian Traffic Management

When designing and constructing track in an urban environment, the walking public must be taken into consideration also. The construction may block foot paths or sidewalks. These pedestrian walkways must be properly rerouted so people do not walk through the construction site and get injured. Pedestrian traffic must be identified and managed during both construction hours and off hours. Understanding that a constructor will need to cut rail and make field welds will help identify clearance zones. Pedestrian bridges that are only a short distance above the pavement can still allow a constructor to have continuity of track construction while maintaining pedestrian flow. Nevertheless, it is advisable to completely separate the construction from the general public whenever at all possible.

13.2.4.6 Pre-Inspection

This activity is a must and may—by identifying interferences and other issues early, when they can be mitigated—prevent them from becoming costly delays during construction. The right-of-way as well as the surrounding community should be inspected by as many disciplines as possible with the object of spotting potential problems with design, construction, and maintenance. This activity could prevent millions of dollars of additional cost and delays by identifying the interferences as well as the associated risks. This will allow for a risk analysis to be performed on the problems and mitigation resolved. This can be as simple as finding a fire hydrant that was not recognized on the drawings or a manhole that is located in the centerline of the track.

13.2.4.7 Baseline Stray Current Report

Stray current is a very serious problem with electrified transit systems. It has nearly destroyed track and severely impacted many transit systems in the United States. There are books written on the subject as well as many lessons learned. The designers should familiarize themselves with as much information as possible prior to making decisions that could affect the longevity of the system. It is a good practice to do a baseline survey of existing stray currents that may be in the ground before any construction begins. This information would offer the utility company(s) time to correct any problems and possibly install cathodic protection on the existing utilities. This survey could be a very important document that could possibly protect the owner and the project from future litigation. See Chapter 8 of this Handbook for additional information on stray current.

13.2.4.8 Contaminated and Hazardous Materials

When excavating for certain trackforms, it is best to understand what is in the ground prior to excavating for embedded track. In urban areas, it is virtually certain that contaminated and hazardous materials will be encountered during excavation, and an action plan should be

prepared so that there is no lost time in production and claim negotiation. These types of materials may fall into the category of "unforeseen" conditions and may stop a job in its tracks. In some cases, it may be prudent to establish a fund of money to cover these conditions and allow the constructor to draw from this fund on a time and material basis. Archeological finds are possible and can also delay a project. A review of the project's environmental clearance documentation will sometimes provide information about where hazardous materials and archeological items might be found. Such information generally should be included in the construction contract documents "for information only." This enables the constructors to have an action plan in place for such situations, thereby minimizing claims and delays.

13.2.4.9 Permitting

A wide array of permits and permissions are required from public agencies before construction can proceed. Clearances are also often required from public utilities and railroads whose facilities are affected by the project. The permits required should be identified well in advance, including who is responsible for getting them. It may take a very long time to get certain permits and to simply assign the obligation to the constructor may delay a project. The designing engineer will at least have the responsibility to verify and identify all permits that are required or may be required. In general, permits that relate to the details of the design should be secured by the owner or designer. Permits that relate to the constructor's means and methods and how those processes affect the community should be obtained by the constructor. In many cases, the design team will have laid the groundwork for those constructor permits.

13.2.4.9.1 Detours

When the project is constructed within a city street, setting up detours and maintaining vehicular traffic can be a full-time job for a significant part of the constructor's team. Synchronizing roadway outages with the construction schedule is always a challenge. Confirming that all the material and equipment needed for a construction activity is actually at the site prior to closing roadways is essential. It may not be advisable to close roadways when certain material is only promised to be on site in time. Permits and advance warning notifications should be the norm. Understanding the municipality's requirements is important also. Some may have a moratorium on cutting fresh asphalt or may not allow detours during certain sporting events or concerts. Some owners may have a restriction on activities performed between Thanksgiving and Christmas, during the shopping season. Pedestrian traffic must be maintained as well, including access to and space for existing bus stops. These matters should be investigated early, and the accepted mitigation measures clearly identified in the bid documents. That way, the constructor will have budget in the bid prices to address these matters and can structure a work plan to accommodate the requirements. The alternative—identification of needs only as they arise—will only result in delays, claims from the constructor, a dissatisfied public, and an owner who is upset with the designer.

13.2.4.9.2 Fire/Emergency Response

Notifying emergency responders is always advisable in order to have a smooth running project. Firehouses and Police departments must know which roads will be closed and how construction will impact their normal activities.

13.2.4.9.3 Dig Safe

The appropriate utility companies should always be notified prior to any excavation. Generally, on any LRT project, there has been extensive coordination with the utilities during the design phase of the project; therefore, most, if not all, of the utilities will have been relocated well in advance of track construction. Nonetheless, this must always be verified. The contract documents must be very specific about who is responsible for the notification to utilities and whether allowances must be made in the project schedule for utility work. It is particularly important to identify any situations where the constructor must work around live utility lines that are to remain in place. Such situations may have a direct effect on the constructor's means and methods and, hence, costs. Leaving these issues up to the constructor can delay projects and incur extra work claims, especially if there are utility interferences. If such interferences exist, time must be reallocated in the schedule to allow for relocating the utilities so that they are not in conflict with the system construction.

13.2.4.10 Scheduling/Planning

This single activity can be the downfall of any project if not performed properly. The persons involved should be professionals at making schedules and planning activities. There should be a detailed understanding of how long it takes to perform activities and which ones will impact the follow-on activities. This analysis ultimately defines the project's "critical path" and is documented in a baseline Critical Path Management (CPM) schedule. Many disciplines should be involved in the creation of this document. This document will identify potential delays and offer opportunities to recover from them. The constructor and the engineer are the drivers behind the CPM. This document should be produced well in advance of construction and periodically adjusted and updated to reflect progress. This document will also serve as a tool for evaluation of claims and change orders. It is strongly recommended that construction should not begin unless everyone involved agrees that the baseline CPM represents the path on which the job will progress.

13.2.4.10.1 Risk Analysis

Beginning a project without performing a detailed risk analysis can be deadly. This risk analysis is simply playing the "what-if" game. There have been volumes written about risk analysis, and many methods can be employed, but the basic concept is simple. The engineer, constructor, or owner will take a certain activity and develop all the things that could go wrong, the probability of that occurring, and have an action plan for each one. If the outcome is too severe, a mitigation process takes place to reduce the risk, such as by doing it another way. An example may be an activity that could cause serious injury or loss of life. The easy mitigation is to take the human out of the equation. The designer should be knowledgeable about the activities that will put too much risk onto the project and identify strategies to reduce that risk to an acceptable level.

13.2.4.10.2 Survey

A topographic and land survey must be performed to support the final design. Under the Design/Build project procurement method, it is sometimes possible for the owner to limit the pre-contract survey to simply identifying the right-of-way and offering design criteria for the constructor to work out the civil engineering details. A word of caution: if the project right-of-way is a former railroad corridor, the real property boundaries shown on the railroad company's "valuation maps" could be grossly out of date. In any event, the right-of-way must be established and clearly marked, including benchmarks and monuments for reference during construction.

This survey may be very elaborate if the documents are brought to the 100% stage. This survey also forms the basis for interferences with utilities or structures.

13.2.4.10.3 Document Control

Document control can be an entire division within a company that has the sole purpose of making sure that all paper is stored and cataloged properly. Security is a very important part of document control. In some cases, redundancy, such as off-site storage of back-up copies, will be employed to address issues such as loss due to disasters such as fires or floods. Electronic storage is a challenge including proper back-up and security. Going paperless is good for the environment, but it must be remembered that security for electronic files can be overwhelming, especially if there is litigation at the end of the project. All electronic material must have very special measures employed so that no one can tamper with these documents. The FRA has identified some methods, but at this writing still requires paper for audits. Organization is the key to good document control.

13.2.4.11 On-Track Construction Equipment

Typically, the constructor will need hundreds of items of construction equipment to build the job and some of that will need to be able to ride on the rails. However, even if the track on the project is standard gauge, on-track equipment that is designed to work on a freight railroad will not necessarily fit the trackwork and clearances of a light rail transit project. See Chapter 14, Article 14.6.1 for additional guidance on this matter. Whatever limitations the track or guideway might place on the constructor's equipment should be clearly identified in the contract bid documents.

13.2.5 Relocation of Utilities

Identifying potential interferences with local utilities prior to construction activities is imperative. The level of utility investigation usually performed for a civil/highway project may provide insufficient detail for an LRT construction project. There are a host of utilities that could be affected by the construction or must be protected during construction. Protection slabs and corrosion control measures may be necessary. Utility companies should be notified early in the design process and continuously kept abreast of the project progress. Pre-meetings (both pre-design and pre-construction) are a great resource, and utility companies appreciate being invited and made part of the process.

13.2.6 Activate Detours

Coordination with the highway department(s) and emergency response organizations is imperative, and the designer should notify these agencies and ask for input on disruption to their system so that appropriate requirements are included in the project procurement documents. Note that some projects will cross multiple jurisdictions, and the requirements of each might well vary.

13.2.7 Quality Process

Maintenance of an acceptable level of quality in the construction is arguably the most important activity for a successful project. Projects and project subcomponents will fail, or at best, struggle

forever without a good quality program (QP) during construction. Quality includes both quality control and quality assurance.

13.2.7.1 Developing the Quality Program

The constructor should be in direct control of the quality of construction. The work should include the first level of inspection, testing, and verification that a feature is constructed with material and workmanship that meet the requirements of the contract. The quality control program should consist of the plans, procedures, and organization necessary to provide inspection, testing, and verification that materials, equipment, workmanship, fabrication, construction, and operations comply with contract requirements. The constructor's quality control plan (QCP) should be submitted, reviewed, and accepted prior to any commencement of work or production of material. This plan should include at a minimum:

- A description of the QCP management organization. This should include an organization chart showing the relationship of the quality control organization to other elements of the constructor's project staff and overall organization.
- Number, classification, qualifications, duties, responsibilities, and authority of personnel.
- Procedures for processing contract submittals.
- Inspection, testing, and verification activities to be performed to ensure compliance with contract requirements, including such activities performed by subcontractors, suppliers, and off-site fabricators as a part of the requisite quality control system of each subcontractor, supplier, and off-site fabricator to ensure compliance with the requirements of the QCP.
- Testing procedures, including recording and reporting test results.
- Format for documentation of the QCP activities.
- A copy of the letter appointing the QCP manager, signed by an officer of the firm, outlining the QCP manager's duties, responsibilities, and authority.

Categories of a good plan should at a minimum include the following:

A. General
   a. Definitions
   b. Responsibilities
   c. Constructor and engineer
   d. Coordination meeting
B. Quality control plan
   a. General
   b. Organization, staffing, and hierarchies of responsibility and authority
   c. Staff
C. Testing and testing plan
D. Control of measuring and testing equipment
E. Completion inspection
F. Documentation
   a. Preparatory phase checklists
   b. Checklist prior to placement of plinth concrete

c. Plinth concrete placement
d. Plinth concrete post placement
e. General trackwork
f. Skeletonized track
g. Track type
h. Welded rail

G. Quality program sign-off
H. Elements of control
I. Management of submittals
J. Material handling
K. Subcontractor and supplier control
L. Control of construction
M. Control of nonconforming conditions
N. Audits and procedures for audits

13.2.7.2 Checklists

Checklists are a valuable document to maintain and make part of the document control system. These are simple reminders of what to look for prior to a phase of construction proceeding. These checklists should be filled out and signed by a qualified person and acknowledged by a supervisor. In some plans, these checklists become "stopping points" if deficiencies are identified. If the deficiencies are not corrected, they could become a non-conformance item that must be corrected prior to acceptance. Checklists should be project-specific since projects vary, with widely different standards and requirements for design, materials, methodologies, and workmanship.

13.2.7.3 Non-Conformance Reports

During construction it is common for the inspector to write non-conformance reports (NCRs) that simply mean that the constructor is deficient with some aspect of the project. Generally, this means that the constructor has violated a specific aspect of the specifications. The NCR must be satisfied prior to acceptance of the project. It is good practice to immediately fix the problem and have the author of the non-conformance violation sign-off that it was completed satisfactorily. An NCR is a serious violation and should not be confused with punchlist items. An NCR will only be written if there is a complete disregard of the performance standards. An example of an NCR would be that the constructor used material that did not comply with the specifications, such as inferior concrete ties or direct fixation fasteners. An example of a punchlist item would be that the strands in the concrete tie protruded more than the criteria. In general, NCRs are non-negotiable whereas a punchlist item may be waived due to negotiating tactics. An NCR would be something that jeopardizes the safety of the system whereas a punchlist item would be a minor infraction. An analogy might be speeding in a school zone as opposed to speeding on a rural Interstate highway.

13.2.8 Reinforcing Steel (Embedded and Direct Fixation Track)

Because of the possibility of stray traction power current, extraordinary measures must be taken with the reinforcing steel used in rail transit concrete structures. The paragraphs below elaborate on those issues.

13.2.8.1 Epoxy-Coated Rebar

If the method of controlling stray current is the "isolation" method, it is necessary to coat the reinforcing steel with epoxy. This adds another level of protection from stray current. Welding epoxy-coated rebar together is not required.

When using epoxy-coated rebar it must be understood that if the epoxy is chipped or compromised, it must be repaired with an approved coating. If epoxy-coated rebar is cut for any reason, the exposed end must be painted with epoxy. If the bars are scratched or nicked in any way, they must be touched up with epoxy paint. Detecting and correcting all such issues before the concrete is poured is a major job in itself. Continuous inspection is usually necessary as the bars are placed as subsequent bar placement may make visual inspection and correction difficult to impossible. Quality programs must address this prior to any concrete being placed.

13.2.8.2 Black (Uncoated) Rebar

Uncoated reinforcing steel should be welded together to ensure a continuous mat, cage, or grid. This is commonly called electrical bonding. If there is one piece of rebar that did not get properly attached to the grid, that piece will become a "hot spot" where stray current will migrate and begin the deterioration of the entire concrete slab or nearby utilities. The entire reinforcing steel system is electrically bonded and connected back to the negative side of the traction power system. As when using epoxy-coated rebar, this method requires intense quality control measures to detect and correct locations where intersecting bars have not been properly bonded. Factory-made welded bar mats can dramatically reduce the amount of field labor and associated costs.

13.2.8.3 Quality, Risk, and Cost

Whichever method is chosen, it is imperative that good quality control systems are in place. Using uncoated steel is somewhat more risky than using epoxy-coated steel. When a risk analysis is performed, it should be apparent that isolating the rail with as many redundancies as practical is a better method. Cost is a factor; therefore, it may be wise to do a cost analysis along with the risk analysis. If the rail is fully isolated, there will be little to no stray current and no appreciable opportunities for failure.

13.2.9 Rail Grinding

Rail grinding is frequently the last major construction activity undertaken by the track constructor. This topic is discussed extensively in other Chapters of the Handbook and will not be repeated here. See the following Articles for additional information:

- Chapter 4, Article 4.2.5.3
- Chapter 9, Article 9.2.1
- Chapter 14, Articles 14.6.2.2 and 14.6.3.8

13.2.10 Track Geometry Verification

Before final acceptance, it is usually required to verify that the track, as constructed, meets all the geometric requirements of the contract. This includes the verification of gauge and superelevation as well as the actual as-built location of the track. Often times, it is proposed to use a "track geometry car" to collect this information. Geometry cars can be a very useful tool for

collecting some of this information, but the raw output data are generally not immediately useful without some analysis. Track geometry cars are generally programmed to detect deviations from a particular FRA track class and the raw output data need to be filtered to gain an accurate understanding of the actual conditions. Plus, the geometry car cannot verify that the actual as-built location of the track is within tolerances of the mathematized alignment shown on the plan and profile drawings. Hence, it is typically necessary to supplement the geometry car data with information collected by traditional manual topographic surveying methods.

13.2.11 Project Close-Out

This activity can take a long time, and this discussion will offer the designer some tips on how to expedite the completion of a project. The project close-out process should actually begin at the very beginning of the project and continue until completed. The methods and procedures for close-out should be as detailed as the construction itself. It can be detrimental to a project that waits until the end to close-out. Usually, the key people will have moved on to the next project, and valuable information is lost or forgotten. In some cases, this activity may take as long as the construction did; therefore, some activities should be continuous and closed out as they come to an end.

13.2.11.1 Clean-Up

Cleaning up the project can be a very cumbersome and overwhelming process. This not only pertains to the physical cleanliness of the structure but also to the organizational aspect of the quality programs. If they are attended to as the work progresses, the end is not so overwhelming. If everything is left to the end, it can be so overwhelming that corners may be cut and details overlooked. This could lead to an owner that is not pleased with the project or that has some concerns with the safety and reliability of the system. In some cases, the clean-up is pushed by politics, and quality is what suffers. The clean-up should also begin with the construction, and, if all is done properly, the close-out is simply a handoff. Checklists can also be used for this activity. A brief example would be as follows:

- Material certifications
- Final quantities for unit price items
- Verification that all subparts of lump sum items are complete
- Resolution of change orders and claims
- As-built drawings
- Resolution of final contract amount
- Resolution of time-related issues
- Timetable for close-out set
- Close-out subcontracts
- Final EEO report
- Special warrantees, insurance, or extended warrantees
- Final quality control documentation is transmitted
- Final invoice with retention payment prepared and submitted
- All equipment and small tools returned
- Temporary utilities disconnected
- Temporary facilities removed
- Final jobsite clean-up
- Site demobilized

13.2.11.2 Safety Certification

Safety certification, which is now an FTA requirement, is a process that begins during final design engineering and extends through the construction process. A safety certification process, managed by safety specialists, will result in certification checklists that must be completed at milestones in the design and construction process. This affects all elements of the constructed project, including trackwork. The constructor should obtain a safety certification checklist from the owner prior to beginning work. As construction progresses, the necessary inspection records needed to satisfy the certification can be collected and the items on the checklist completed.

13.2.11.3 Documentation Retention/Storage

Choosing the right way to store all the documents that are produced during the course of a project is paramount to the success of the project. Back-up systems, security systems, access systems, and a cataloging system must be chosen, and, once a system of document control becomes active, it is difficult to change during the course of the project. Many agencies are going "paperless," and this presents some challenges. E-mails present special challenges and their storage is very important. Directives that result in major changes in a project, potentially costing someone millions of dollars, have been given through an e-mail.

Having a system that cannot be altered at a later date is a must, especially in the event that there is litigation at the end of the job. When choosing a system for proper storage of documents, it may be wise to use a consultant that is well versed in the subject and can suggest ways to protect against the types of document fraud that could take place. If paper systems are chosen, they also must have certain safeguards in place. Scanning paper into electronic form is a good back-up. Information management technology is constantly evolving, and research to find the best system for a particular project is necessary to ensure that all documentation is accurate and accounted for.

13.2.11.4 Project Record Documents ("As-Built" Drawings)

The project record documents include not only updated versions of the original contract drawings but also drawings, shop drawings, photographs, field sketches, revised specifications, correspondence, and any other information that provides a complete and accurate record of the project's end product. At the end of the project, these documents must be submitted to the owner. All approved change orders or other design modifications, punchlist signoffs, and non-conformance report resolutions should be included in this final document. Submittal and acceptance of the project record documents should be linked to final payment.

This activity should actually begin at the beginning of construction and continue until all trackwork is built and accepted. If accurate records are kept in an organized fashion during construction, completion of project record documents can be a relatively simple task. If not, much of the needed data may be difficult or impossible to resurrect, and final payment to the constructor could be jeopardized. It may be very difficult to resurrect information after the track or track components are embedded in concrete or otherwise concealed; therefore, every change of field conditions or change to the bid documents must be recorded as they occur and not after the fact. It's also necessary to keep up with the paper documentation trail. For instance, mill certifications for furnished materials are typically required by the specifications, and missing certifications are

cause for a non-conformance report (NCR). In general, all NCRs must be signed off on before final payment or retention is released. Waiting until the end of the project to ask suppliers for mill certifications and then waiting for the suppliers to come up with them would therefore affect final payment. Since the suppliers may have long since been paid for their products and services, they may have little incentive to follow through with deferred paperwork.

13.2.11.5 NCR Sign-Off

An NCR is a formal document that is produced during the course of construction. An NCR is not just for physical construction non-conformance, it can be related to specifications or shop drawings. Anything at all that does not conform to the original documents should have an NCR assigned to it. These reports are much stricter in nature than a simple punchlist item. Punchlist items can possibly be informally negotiated; however, an NCR must have formal documentation of the corrective action that was taken before it is officially signed off on. There should be an NCR log that contains where, when, and what is not conforming to the contract. There should be a column for resolution as well as formal sign-off. This report is a living document, and each item must be resolved before final payment is made. Every project meeting should have an agenda item that addresses NCRs. These items must be addressed as they occur and not be “saved up” until the end.

13.2.11.6 Close-Out

Finishing the job can be a very cumbersome task, especially if “minor” activities are left until the end of the project. On most construction jobs, close-out may be completed by personnel that had little or nothing to do with the actual construction. The key personnel have very often moved on to the next project. It therefore may be difficult to resurrect information required to satisfy the owner that the project was built according to the contract drawings and specifications.

Storage of all documents must be discussed if not identified in the contract. Will electronic storage suffice or must everything be in paper form? What security will be introduced so no changes can be made after the fact?

This is where a good quality control program can pay off. Periodic meetings, quality control, and full completion of individual tasks during the project will make close-out much easier for the constructor, the designer, and the owner.

13.3 CONSTRUCTION ISSUES FOR DIFFERENT TRACKFORMS

13.3.1 Construction Issues for Ballasted Track

The purpose of this article is to offer procedures for the construction of ballasted track for a light rail system. Much of this is also applicable to heavy rail and freight and passenger and commuter railroads; however, due consideration must be given to the increased loads and dynamic forces of those other modes.

13.3.1.1 Construction Concept

It is important to understand how the track will be constructed in order to ensure continuity and coordination with other constructors. It is also important to understand some basic production rates that may be possible. This can be very important when putting together a baseline

schedule. What is certain is that the constructor who is doing the trackwork will have appreciably different concepts about how the project can be constructed than those of the designer. Unless there is some definite reason why some portion of the work must be constructed using a particular method or constructed in a particular sequence, the constructor should be given appreciable latitude concerning his "means and methods" so long as the end product meets the requirements. Forcing the constructor to do otherwise will add cost to the project without necessarily adding value or quality.

13.3.1.2 Construction Activities

The fundamentals of the art of building ballasted railway track have not changed much since the early days of railroading. We have simply developed better equipment to handle the work. Ballasted track is usually initially constructed in a "skeletonized form," meaning the two rails have been fastened to the cross ties at the proper gauge distance apart, but no ballast has been placed. This skeleton track is roughly aligned and then the ties are surrounded by ballast stone that will hold it in place vertically and horizontally. The track is then raised, brought to final alignment, and the ballast stone tamped (compacted) to hold the completed track in alignment. The discussion below will explore this basic construction process in more detail. This is not intended to be an all-inclusive list and is primarily directed toward designers so they may have a better understanding how their designs can affect construction.

13.3.1.2.1 Surveying

A competent survey party must be incorporated into the project. It is generally advisable to pre-qualify at least the survey party chief since many highway survey parties have not been trained in railway stakeout, particularly turnout geometry and spiraled curves. Pre-qualifying and training survey parties are essential to prevent misalignment and elevation problems. Some of the surveying issues that often arise on projects include the following:

- Spiraled curves can be particularly troublesome for an inexperienced surveyor. This becomes particularly critical in direct fixation and embedded track where the surveyor must be able to set grade not only for the low rail of the curve, but also for the outer edges of the track slab formwork.

- The survey party must understand all the control points of a turnout, what they are properly called, and exactly which points the track constructor needs to position the track materials on the ground. Confusing the point of intersection of a turnout (PITO) with the point of switch (PS) can be devastating to the project. Very often, the constructor doesn't need the PITO for construction and would much rather have the surveyor provide reference stakes for the PS and the point of frog instead.

- The survey party must understand the difference between the edges of the subballast layer and the edges of the subgrade. If the subgrade is staked and constructed to the width shown on the drawings for the top of subballast, embankments will be too narrow to both support the ballast section and provide a walkway at the ballast toe.

13.3.1.2.2 Handling Material

All track material must be handled properly and the constructor should prepare and submit a material handling plan for every component. This plan should include the equipment to be used and the method of rigging as well as the type and size of the dunnage separating the layers. A storage area plan should be submitted showing the location of all types of material as well as service roads and equipment staging. Each item must be packaged correctly for the equipment and rigging that will be used. Some issues with handling material include the following:

- Continuous welded rail (CWR): CWR must be handled correctly by an experienced and pre-qualified equipment operator. If handled incorrectly, the CWR can be bent, flipped, or damaged. A simple nick in the rail can cause a break when the rail is in tension in cold weather. When CWR is in a stockpile, the dunnage between layers must be vertically in line so no bending stress is imposed. Improper placement of dunnage can cause permanent vertical bends in the rail that must be cut out. CWR is very flexible, and it is tempting to laterally shift it more than it can handle, which can cause a permanent lateral bend that must be cut out.

- Cross ties: There have been many examples of ties failing long before their expected service life due to improper handling during construction. Concrete ties must be stacked with dunnage in the rail seat area so as not to cause bending stresses. Wood ties are typically bundled in numbers that can be handled by the equipment used. If bundles of ties are stacked, then the dunnage used must be sufficiently thick so that forklift equipment can get between stacks without banging into the ties.

- Packaging: How material will be handled has a direct effect on how it should be packaged. Even in the case of owner-furnished materials, it is advisable to have the constructor liaise directly with the supplier on issues of packaging and delivery schedule since that can save time and money for all parties and also reduce the possibility of damage to the products. “Other track material” (commonly abbreviated as OTM), such as rail fastenings and fasteners, can be placed in crates; however, the crates must be substantial enough to accept the equipment doing the unloading and the weight of the material inside.

- Long-term storage: Even though track materials are intended for outdoor use, they can deteriorate in storage if not stacked properly. The corrosive effects of acid rain can be particularly damaging to closely stacked rail. Air should be able to circulate around the rails so any moisture can easily drip off and evaporate. The usual practice of tightly stacking rails with the bases touching can hold moisture in the stack and initiate corrosion, particularly if falling leaves accumulate in the spaces between the rail webs so that snow and ice is held against the rail base and web. If rails will be stored for more than a year or two, consider spacing the rails in each tier ½ inch [about 1 cm] apart so storm water is more likely to drain through. Groove rails should always be stacked with a slight pitch from one end of the pile to the other so that storm water will not lay in the flangeway. Covered (but not necessarily indoor) storage can go a long way toward keeping track materials in good condition.

13.3.1.2.3 Underground Systems Construction

Even once earthwork has been completed to create the trackbed, it is not yet time to begin track construction. First, prior to installing the subballast, the constructors responsible for the installation of underground electrical ductwork and manholes should be required to complete as much of their work as possible. This, in some cases, includes direct burial cables. Similarly, catenary pole foundations and the underground portions of other vertical construction (such as the foundations for signals) should be installed prior to the placement of subballast. If these items are deferred until after track construction has commenced, the subballast and ballast will be disturbed and contaminated with subgrade materials, very likely creating locations in the track that will be maintenance headaches for decades. To avoid these problems, it is highly desirable that the contractors who will be constructing these underground systems elements be on board and mobilized before track construction commences.

However, in many instances, the train control systems, traction power systems, and other electrical networks that will require underground conduits have not even been designed when the civil constructor begins grading and excavation. This very often leads to trenching for underground ducts well after the track is constructed. Any such excavation within the trackway after the track has been built will cause problems. If the subballast is compromised with inappropriate backfill material or an underlying geotextile layer is punctured or destroyed, it is very difficult to restore the original integrity of the track's substructure. Settlement of the track and contamination of the ballast, both of which interfere with proper drainage, will result and ultimately reduce the service life of the track structure. For these reasons, the track designer should strongly encourage the construction of all underground ducts prior to the placement of the subballast.

For just this reason, some transit agencies utilize separate "systems elements" contracts to install as much of the underground ductwork as possible prior to the construction of tracks. The empty ducts and manholes are then turned over to a systems contractor for the installation of cables and final connections to signals, bungalows, and other above-ground systems infrastructure.

If, as is often the case, some of these systems elements simply cannot be installed ahead of the track construction, it is essential that the contract that will install them includes specific specification language concerning how the track subballast and ballast are to be restored, including compaction and tamping requirements. Even then, it is also often necessary to take extra effort to impress upon the constructors of these systems, as well as the construction inspectors monitoring their work, the absolute importance of correctly restoring disturbed portions of the track. Specifications notwithstanding, in the absence of close oversight by the trackwork side of the project (including the designer, the track constructor and any construction manager overseeing the track construction), it is highly probable that the track will not be properly restored after underground systems construction.

13.3.1.2.4 Placement of Subballast

The placement of the subballast layer is typically performed using a spreader box that defines the edges of the aggregate and screeds it to proper grade. So that the finished elevation of the subballast is within tolerances, it is essential that the subgrade on which it is placed is properly shaped and compacted. Subballast should not be placed on subgrade that is rutted, crisscrossed with trenches full of uncompacted backfill, frozen, or muddy. Once the subballast has been

placed, it needs to be compacted to the specified density and the finish elevations verified. Compaction is usually done with a vibratory roller, the larger the better. Once placed and compacted, the operation of rubber-tired vehicles over the subballast should be extremely restricted. It is essential that any ruts or other damage to the subballast be repaired prior to it being covered with the initial layer of ballast.

13.3.1.2.5 Layout Rail and OTM

When to layout the rail and other track material is a decision that must be well thought out. Laying out the CWR too early can result in damage to the rail. If other contractors subsequently need to access the right-of-way, they might drive their equipment over the loose string of rail. If the rail is not firmly supported, their trucks may bend or damage it. If one of those trucks flips the rail, someone who is standing near the rail ¼ of a mile [400 meters] away could be seriously injured or even killed. If other contractors attempt to do excavation under the rail, it could be damaged by the excavator bucket. Laying out clips, bolts, spikes, or any other OTM ahead of the track construction can also be cause for damaged, lost, or stolen material. These activities must be well thought out and incorporated in the action plans. Each activity must have assigned equipment, tools, material, and timing.

13.3.1.2.6 Spread Initial Layer of Ballast

Raising track up through dumped ballast places significant stresses on the rail fastening system. So as to minimize those stresses, it is good practice to limit the actual raising of the track. This is achieved by placing most of the ballast below the ties prior to the construction of the skeletonized track. This layer of "bottom ballast" is placed and compacted on the subballast, with the top surface about 2 to 4 inches [50 to 100 mm] below the final elevation of the bottoms of the cross ties. This is easily accomplished using conventional road building equipment such as a spreader box and bulldozer or motor grader. Once this bottom layer of ballast is brought to the proper elevation, a drum roller should thoroughly compact it so that the ballast particles are firmly interlocked and the surface is planar and unyielding. This interlocking of ballast is critical for the stability of the track under thermal and dynamic forces.

Once the bottom ballast is placed and compacted, care must be taken not to rut the ballast surface causing the ties of the skeleton track to have an uneven bearing.

13.3.1.2.7 Place (Bed) Cross Ties

Concrete Ties: Placing concrete ties requires equipment since a concrete tie can weigh up to 850 pounds [385 kg]. There are a number of types of equipment to handle these ties:

- There are attachments to standard excavating equipment that can handle eight ties very easily and space them correctly.
- There are spreader beams with hanging cables spaced to match the correct tie spacing.
- There are "boom trucks" (sometimes called "log trucks") equipped with grapples that can handle concrete ties.
- A front end loader with forks can be used.
- Track-laying machines have conveyance systems to correctly place and space the ties on a prepared layer of ballast.

Whichever method is chosen, it is imperative that the concrete ties lay flat and do not have ballast bearing at their centers as this will cause the ties to crack later in the process. Some ballast screeding equipment compensates for this by creating a slight concave depression along the center of the track. There should be full bearing between the bottoms of the ties and the bottom ballast layer for about 14 inches [35 cm] each way of the tie's rail seats.

Wood Cross Ties: Wood ties, being much lighter than concrete ties, can be handled much more easily, including manually by workers using tie tongs. A log truck with a grapple is a popular method for handling wood ties. The ties can be roughly spaced on a prepared ballast layer by the log truck and then properly spaced using tie tongs. Wood ties are more "forgiving" than concrete ties when placed on uneven surfaces.

Other Types of Ties: Steel, plastic, and composite ties can be handled and placed much like the wood ties. They are usually lighter in weight, and therefore more pieces can be moved with the same equipment. They must still be placed at the correct spacing and alignment on the prepared subgrade or ballast layer.

13.3.1.2.8 Set Up Line Side

If wood or plastic ties are chosen, they can be pre-plated. Pre-plating carries some risks and challenges in order to create the gauge and cant correctly based on the size of the rail. Using the same dimensioning for 115-pound rail as 136-pound rail will not work and will create either wide or tight gauge when the rail is set. If pre-plating is not chosen, one plate needs to be set toward one end of the tie in the correct position. This activity is called "setting up line side." This offers a control when setting the rail into the plates. Quality control is an important part of getting this right.

13.3.1.2.9 Gauge Track

If using concrete ties or steel ties, then the fastening system is already incorporated in the manufacturing process and track gauge is generally fixed. In most cases, only spot checking or gauge is needed. Some concrete cross ties allow a small amount of gauge adjustment by changing out the plastic rail insulators beneath the rail clips. When using timber ties, the second rail must be correctly gauged relative to the first rail after it has been secured. The ties can be pre-plated using a jig to set the plates to correct gauge, but even then, the gauge should be verified during rail laying.

In general, it is not necessary to check gauge at every tie. Assuming the use of new rail that meets specifications, gauging every fourth tie is usually adequate in tangent track and flat curves. Sharper curves need to be gauged at closer intervals. Extremely sharp curves may need to be gauged at every rail fastener even if the rail is precurved. When gauging track, it is imperative that both rails are seated properly into the plates with the correct cant. Setting gauge with the rails canted wrong will cause tight gauge after the rail becomes seated. Just one tie plate that is backward can cause incorrect gauge in 20 feet [6 meters] of track.

13.3.1.2.10 Pre-Line and Clean-Up

After the track has been constructed in a skeletonized form, it is advisable to pre-line the track to within 1 inch [about 2 to 3 cm] or so of theoretical. This helps follow-on operations and will ensure the right amount of rail is in the track. Making dramatic alignment adjustments after the ballast has been placed can introduce compressive or tensile forces in the rail, which could prevent the

track from ever staying in proper alignment. Then, the only way to fix the problem is to cut the rail and add or subtract rail as necessary. The same sort of problem can occur if the top of the bottom ballast layer is not a reasonably uniform distance below the final track profile.

A general clean-up of the track prior to ballasting will prevent both valuable track material and debris from becoming lost in the ballast.

13.3.1.2.11 Dump Top Ballast

The next step is to introduce more ballast into the skeleton track structure. The ultimate goal in this step is to have a cross section that has the ballast up to the top of rail and a robust shoulder. This "top ballast" is generally placed up to the top of the rail and continues at that elevation to a point about 12 inches [30 cm] beyond the ends of the ties, where it slopes down to the trackbed. If the bottom ballast was placed correctly, more ballast than this should not be required to achieve the finished ballast cross section.

During the placement of the top ballast, it is imperative that no equipment apply pressure on top of the ties even after the ballast has been placed. This will dislodge the fasteners if the tie is not 100% supported underneath. There are several methods for placing ballast into the cribs and shoulder of the track, but, in general, only on-track equipment should be used. Top ballast can be placed with standard bottom dump railroad hopper cars or hy-rail dump trucks. Some contractors have low platforms with rail wheels that are large enough to accommodate a standard dump truck loaded with ballast. These are towed by a small locomotive to ferry dump trucks to and from the ballast placement site.

If site conditions permit, ballast can also be placed from the side of the track with standard heavy construction equipment such as front end loaders. However, rubber-tired vehicles without hy-rail gear should never be driven on top of the track. Too much damage can be done; for instance, the rail can be gouged. If a nick is in the rail, it is considered a defect and therefore must be replaced. The ties could be damaged or dislodged from the fasteners. The ties could crack under load, but the damage might not be apparent for years.

13.3.1.2.12 Raise, Line, and Tamp Track

This step in the production of a good ballasted track requires some specialized equipment specifically designed for raising the track out of the ballast up to the correct grade, positioning the track laterally to exact alignment, and then tamping the ballast. So as to minimize strain on the rail fastening system, this should be done in at least two lifts, each not more than 3 to 4 inches [25 to 75 mm], with the final lift being no more than about 1 inch [25 mm]. All automatic tampers work on the alignment relationship between a light projector and a shadow board receiver positioned a specified distance away. This geometric relationship is what produces finished track within very tight tolerances.

The ballast should be tamped under the ties for 14 inches [35 cm] on either side of each rail but not the middle of the tie. Ties should never be tamped in the middle except at road crossings, where the ballast is captive. Over time the ballast will migrate to the shoulders, and, if the middle of the tie was tamped, the tie becomes "center bound." This will cause the tie to crack in the middle, especially concrete ties.

Extra care must be taken when using these machines so they do not drop the tamping tools on top of a tie, damaging it. It is also necessary to set the depth that the tamper's work heads go into the ballast. If they are set too high, they will damage the bottom of the tie, especially concrete ties. If they are set too low, they will not tamp the tie tight enough to hold the elevation correctly. The correct setting is when the top of the tamping tool is no more than 1inch from the bottom of the tie. An experienced tamper operator is an absolute must or serious damage could happen that may not be detected until the track buckles or the track settles prematurely.

13.3.1.2.13 Dress and Broom Track

This activity requires a ballast regulator, a rail-mounted piece of equipment with the capability to transfer ballast from one side of the rails to the other in order to produce the desired cross section. This equipment also has the ability to "broom" the track, sweeping the ballast to an elevation equal to the top of the tie. Using the "wings," it can shape the ballast shoulder, which accounts for 35% of the holding power when the rail is above or below the neutral temperature creating compressive or tension forces in the rail. Ballast can be dressed manually using shovels, but, since that process is labor-intensive, it is used on only very small projects. The important aspect is that a consistent cross section must be maintained throughout the length of track. One small area that is weak on ballast shoulder will be a high-risk area for a track buckle during hot weather. Very often, the track vibration caused by a moving train is the instigator of a track buckle that will occur suddenly and only a very short distance ahead of the train. Derailment is the usual result, so track buckling is a very serious issue.

13.3.1.2.14 De-stress and Make Closure Welds

De-stressing continuous welded rail is an activity that could jeopardize the stability of the track structure if done incorrectly. Any owner of track who is subject to FRA oversight must have a standard procedure for de-stressing CWR and maintaining the neutral temperature of the rail. Neutral temperature is the temperature at which a rail of fixed length is neither in compression or tension. Determining the neutral temperature is specific to every region and every owner. The neutral temperature could be 95 degrees F [35 degrees C] in Connecticut and 130 degrees F [54 degrees C] in Arizona. AREMA offers guidelines to determine rail neutral temperature based on the highest and lowest rail temperatures experienced at the project site over the previous 50 years. This neutral temperature must be established at the onset of a project. It is also important to understand that the neutral temperature of the installed rail will possibly change if the track is realigned or the profile elevation is changed. That is why thermal de-stressing of the rail should be done only after the track has been brought to the final line and surface and all the ballast is installed and dressed. De-stressing with insufficient ballast in track will not achieve the goal and serious problems will happen later.

The amount of compressive force in the rail will vary with the temperature, the cross-sectional area of the rail, and the constrained length of the rail string. A good, well-built, ballasted track can withstand about 187,000 pounds [about 182 kilonewtons] of compressive force. Beyond that, the track could buckle. To avoid this, AREMA says that CWR should generally be anchored at a neutral temperature that is not lower than 50 to 70 degrees F [about 30 to 40 degrees C] below the maximum expected rail temperature. The rail can easily be as much as 40 degrees F [about 22 degrees C] above the ambient air temperature on a hot sunny day, so it is important to understand that for track exposed to sunlight, the daytime rail temperature is virtually never the same as the ambient air temperature in the shade.

Rarely will the rail naturally be at the optimal neutral temperature when it is time to anchor it in place. It is therefore necessary to adjust the rail so it will be in a zero-stress condition at the target rail temperature. As an example, if 1500-foot [457.2-meter] lengths of CWR are to be anchored at a desired neutral temperature of 95 degrees F [35 degrees C], but the actual rail temperature is 40 degrees F [4 degrees C], the constructor must simulate a rail temperature of 95 degrees before the rail can be permanently attached to the ties. This is done by calculating the gap that is needed between rails at their present temperature so that, when they are heated to the desired neutral temperature, the gap will be exactly closed. The formula (using U.S. traditional units) to determine this is the following:

$$G = 0.000078 \times L \times \Delta T$$

where

G is the gap between strings of rail measured in inches,

L is the length of the rail string measured in feet, and

ΔT is the change in temperature in degrees Fahrenheit.

When using S.I. units of measurement, the formula is the following:

$$G = 0.117 \times L \times \Delta T$$

where

G is the gap between strings of rail measured in millimeters,

L is the length of the rail string measured in meters, and

ΔT is the change in temperature in degrees Celsius.

Solving the equation for the parameters stated above determines that a 6.4-inch [163-mm] gap must be left between strings at 40 degrees F [4 degrees C] so that when the rail is heated to 95 degrees F [35 degrees C], the gap will be exactly closed. However, a dimensional adjustment must be made to account for the rail welding process used to make the closure as follows:

- If thermite welding will be used to connect the two strings of rail together, add 1 inch [25 mm] to account for the thermite metal. Hence, the total gap would be 7.4 inches [188 mm]. Some thermite weld kits may require a different dimension, so it is important to consult the kit manufacturer's printed directions.
- If a portable flash butt welder will be used, it is necessary to subtract an amount from the gap to account for the loss of metal as the rail ends are forged together. Typically, the loss would be 1.5 inches [38 mm], so the actual gap would be 4.9 inches [124 mm]. Some welding equipment produces "low consumption" welds that use much less metal, so it is important to understand the equipment being used.

The calculations in the example above have been carried out to a precision of 0.1 inch [2.5 mm], but the calculated gap is typically rounded to the nearest ¼ inch [6 mm] increment. Since ¼ inch [3 mm] of rail movement over the length of the CWR string equates to only about a 3 degree F [1.7 degree C] difference in neutral temperature, that is sufficiently close tolerance for practical application. It's also consistent with the practical tolerances for field measurements and marking.

In this example, the rounded answer is 6 ½ inch [165 mm]. If the work is being done using S.I. units of measurement, the final answer might rationally be rounded to the nearest half-centimeter.

Note that the calculations are dependent on the length of the CWR string and the temperature at which the measurement was taken. Strings will never be exactly some even number length, such as 1500 feet. Even if they were some exact known length at the time of welding, things happen that could have reduced the string length, such as cropping off handling holes on the ends of the string. Therefore, each string length and the temperature at the time of measurement should be measured once the rail is laying loose on the cross ties or rail fasteners. A rolling measuring wheel is sufficiently accurate for this purpose.

Once the movement of the rail to simulate neutral temperature has been determined, the rail is marked at its quarter points to monitor rail movement and thereby ensure that proper movement is consistent throughout the entire string. For the aforementioned 1500-foot CWR string, each quarter is 375 feet [114.3 meters]. To ensure that the rail is moving uniformly, apportion the 6 ½ inches [165 mm] of total rail movement into the rail's quarter lengths. Mark the desired movement at each quarter point on the rail, concurrently making matching marks on the rail and the rail fastener or a cross tie beneath it. Therefore, at the first quarter point there should be 1 ⅝ inches [41 mm] of movement, at the second quarter there should be double that, or 3 ¼ inches [82 mm] of movement (i.e. ,1 ⅝ inches + 1 ⅝ inches). At the third quarter there should be three times the movement at the first quarter point, i.e., 4 ⅞ inches [124 mm], and at the last quarter the full 6 ½ inches [165 mm] of movement.

Before marking these measurements on the rail, it is essential to first relieve any residual internal force in the rail. This requires unclipping the rail from the cross ties (or rail fasteners) and, by using rollers (beneath the rail) or rail vibrators (attached to the rail), eliminate any friction that is between the base of rail and the rail seats. This is a very important part of the procedure. Only once the internal forces have been released should the calculated movement be physically marked at each quarter point.

The rail length is then adjusted to close the gap and achieve the desired movement at each quarter point. This is typically done by beginning at one end, where the string has already been attached to a previously anchored string. Adjustment is performed by either heating the rail or by pulling on the ends of the rail or a combination of the above, typically pushing the string toward its free end. Various types of equipment are used for both heating and pulling. Whether heating or pulling the rail, the quarter point movement must be achieved or the de-stressing processes will not be completed properly and a higher risk of a track buckle is prevalent. Once the predicted quarter point movement is achieved, the desired gap is achieved by cutting the free end, if necessary, to match a fixed point such as another previously anchored rail or a special trackwork layout. The rail fastenings can be reinstalled and the closure weld installed.

The procedure above is generic, and some owners may have slightly different procedures or formulae. It is therefore advisable to consult each property's CWR plan. For example, some track owners prefer to do all of the calculations in inches and therefore use a coefficient of expansion of 0.0000067 in the U.S. units version of the formula. Some owners will simplify the process for field personnel by providing tables that may require extrapolation.

It is essential that the two rails of a track be anchored at very nearly the same neutral temperature. A tolerance of plus or minus 5 degrees F [3 degrees C] is usually specified.

13.3.1.2.15 Install Insulated Joints and Other Appurtenances

It may be common practice for some contractors and agencies to install insulated joints after the rail has been de-stressed. If this is the case, it is extremely important to not add or subtract rail because this will change the neutral temperature. Each field thermite weld introduces 1 inch [25 mm] of rail into the system. This must be accounted for when welding in insulated joint plugs. If more length is introduced, the risk of a track buckle increases. Always keep in mind that a track buckle will very likely cause a derailment.

13.3.1.2.16 Clearing Ballast from under the Rails

Traction power return current in the rails is searching for ground. Some amount of the current will follow routes other than the rail in an inverse relationship to the electrical resistance of that path. This is stray current. The ballast is a slight conductor. Wet ballast is even more so and muddy ballast can be much worse. It is therefore important that ballast not touch the rail. If ballast continues to touch the rail, the track-to-earth resistance test will generally fail. Therefore, the ballast in the cribs between the cross ties must be removed from under the rail, usually to a minimum clearance of 1 inch [25 mm] between the top of the ballast stone and the underside of the rail or any other metallic part of the track structure.

This is commonly called “poking” the ballast and can be achieved by using a track shovel to push the ballast from beneath the rail. Some contractors have adapted other track construction equipment (such as a rail anchor applicator) to do most of the work of pushing the ballast out from under the rail, but manual methods of final clean-up and disposal of surplus ballast are usually necessary. Poking the ballast is not necessary on non-electrified railways; therefore, unless it is clearly called out in the contract documents, the track constructor may not realize it is a requirement and may demand a change order to cover associated labor and equipment costs.

Another activity that can provide better insurance of passing the electrical isolation testing is to use compressed air to blow any ballast fines off the rail fastenings. Ballast fines, when mixed with rainwater, can result in a “paste” that coats the rail fastenings and provides a conductive path to ground.

13.3.1.3 Some Lessons Learned the Hard Way

There have been some very important lessons learned during the design and construction phases of a light rail system. It can be a valuable tool to understand during the design phase of a project. We have an obligation to learn from our mistakes and to share them with the industry. The ultimate user of these systems must be safe, and the designer and constructor have the responsibility to deliver this to the owner.

- **Placing ties on uneven surfaces:** When ties are placed on an uneven surface in a “center bound” condition, they may break under the weight of the rails and track construction equipment, particularly ballast trains and rail trains, if used. Concrete ties are very susceptible to this. On more than one project, hundreds of concrete ties had to be replaced.

- **Not poking ballast:** If the ballast is touching the underside of the rail, the electrical isolation test will fail. Failure of the test is typically cause for an NCR. As mentioned above, the ballast must be removed from under the rail.

- **Using ballast with excessive fines:** When ballast comes from the quarry, there may be a certain amount of fine particles remaining. If the fine particles are excessive, they may block the proper drainage of the ballast section, and over time this will cause premature degradation of the ties and the track structure as a whole. Fines can also make it more difficult to achieve electrical isolation. It is imperative to follow proper specifications for delivery and handling of ballast. When ballast is stockpiled, fine particles will migrate to the bottom of the pile. If these fine materials are loaded into transporting trucks or cars, they become part of the track structure. If the stockpile methodology is used, the bottom 4 to 6 inches [10 to 15 cm] of the pile may need to be sacrificed in order to ensure the integrity of the track structure.

- **Not documenting de-stressing:** When de-stressing rail, there must be good documentation in the form of paperwork as well as markings on the rail. Quarter point markings, including the supervisor's initials, the date, the rail anchoring temperature, and the actual movement should be done in permanent paint. Not having proper documentation will lead to speculation as to whether it was done properly. These records must be maintained and passed on to the Maintainers after the system begins revenue service. If an incident happens sometime later and rail de-stressing is suspected as a cause, the absence of documentation could lead to unpleasant litigation.

13.3.2 Construction Issues for Direct Fixation Track

The purpose of this article is to offer procedures for the construction of the Direct Fixation Trackform. Many procedures have been used, and many have failed. This discussion will identify the advantages and disadvantages of those methodologies and offer a road map to the successful completion of direct fixation track.

13.3.2.1 Design Concept

The designer typically does not have responsibility for decisions pertaining to construction methods; however, it is important for the designer to understand the construction difficulties and pitfalls that could be inherent in certain designs. It is also important to understand that constructors have certain means and methods that they have used on similar projects and are therefore comfortable with, as they expect the same outcome. This article highlights issues that may mean the difference between a success and a failure in track construction and maintainable versus hard-to-maintain once it is in operation. Direct fixation track is a very viable design and easy to maintain if it is built properly. This article will offer the designer some guidance for producing the preferred system that will last for many decades.

13.3.2.1.1 Plinth

The word "plinth" simply means a pedestal. In the context of direct fixation track construction, it is a support platform for the rail and other items necessary for a train or LRV to travel upon a guideway. A plinth can be constructed on top of a slab on grade (including a tunnel invert) or on an elevated concrete structure. Use of plinths is particularly well suited for "top-down"

construction methods since it is placed after the invert is poured, and the fastening devices or inserts are surrounded by the plinth concrete.

13.3.2.1.2 Inserts in Invert

When inserts are placed in the invert or slab concrete, the most direct construction method is "bottom up". The anchor inserts for the direct fixation rail fasteners are generally placed by drilling into the base concrete and installed using cementations grout or epoxy. The inserts can sometimes be supported by templates while the slab concrete is placed. With either method, the inserts must be normal to the top surface of the concrete surface (with some tolerance) so that the assembly isn't loaded eccentrically when the anchor bolts are tightened.

13.3.2.2 Rail Cant and Superelevation

The following is a discussion and definition of cant, unbalance, superelevation, and cant deficiency from a constructor's perspective. It is important to all parties involved that they use the same terminology throughout the project. Many get confused when terms such as "rail cant," "cant deficiency," "superelevation," "cant," "unbalance," "underbalance," "surface," "elevation," "line," and "alignment" are used. These terms are listed and explained below:

- In North America, "rail cant" and "cant" mean the same thing. This is the relationship between the plane defined by the tops of both rails and the vertical axis of each rail. Most often, the rails are pitched inward at a 40:1 slope. Why the rails are canted is discussed in other chapters of this Handbook, but what is important during construction is making certain this relationship is maintained throughout the track system, including tangent, curved, and superelevated track. When using standard timber or concrete ties, getting the rail cant correct is automatic. It is more difficult to achieve in direct fixation track since the rails are anchored to the invert independently. Similar issues can occur in embedded track, depending on the trackform. When laying rail in extremely sharp curves, the head of the rail may curl toward the outside of the curve and result in loss of proper rail cant. Attempts to force the base of the rail down into the rail fastenings can damage both the rail and the fastenings. In extreme cases, it will be necessary to precurve the rails, including cambering, so they sit in the rail fasteners with the correct cant.

- "Superelevation" (which is sometimes called "cant" in other parts of the world) is a stand-alone term that describes the relationship between the two rails as one is raised higher than the other. In general, the constructor is concerned only with actual superelevation (Ea) as called out on the contract drawings. Unbalanced superelevation (Eu) is solely the responsibility of the designer and does not represent anything that the constructor must build.

- "Surface" and "elevation" are used together and generally mean the same thing with the exception that "track surface" usually includes consideration of cross level differential between the two rails. Track elevation is the vertical position of the top of one rail. In most cases, it represents the elevation of the inside or low rail in a curve; however, some properties rotate the track about its centerline. This is sometimes used in tight clearance tunnels. If this method is used, it must be clearly identified on the construction drawings. Most Contractors will use the term "surface," and most engineers will use the term "Elevation." A meeting of the minds would be a wise choice prior to starting a project.

- "Line" and "alignment" mean the same thing. This is the physical path that the track will follow. It is generally based on the centerline of track or the center distance between both rails based on the properties property's standard gauge. In cases where the track gauge is widened (or narrowed) in curves, the contract documents must designate which rail will remain at a constant horizontal distance from the track centerline.

13.3.2.3 Preparatory Work

Preparing for design and construction is as important as construction. Answering all the "what-if" questions is absolutely imperative to the success of any project. Once the construction begins, it should continue to the end without interruptions. Discontinuity is the constructor's nightmare. The following discussion offers a list of activities that should be resolved prior to beginning construction.

13.3.2.4 Top-Down Construction (Recommended)

This article is not intended to be a procedure for the constructor to follow. It is meant to inform the designer of certain activities that should be followed. The constructor should present their procedure to the designer prior to construction commencement, and the designer should be prepared to present "what-if" scenarios that could result in unacceptable quality. Those scenarios should always be resolved before any construction begins.

13.3.2.4.1 Check Guideway

Once the guideway or invert slab is completed, it must be prepared for the placement of more concrete on top. This may require water blasting to remove any laitance or curing compound in order to achieve a good bond between both sections of concrete. It would be advisable to inspect the surface of the guideway or structural slab and correct any defects.

Dowels (also known as "stirrups" or "rebar hoops") are reinforcing steel that project out of the base slab and provide an anchorage for the reinforcing steel in the plinths. The stirrups must be installed in line with the rail. Traditionally, they have been installed parallel to the rail, but it is actually advantageous to install them perpendicular to the alignment. This will allow for the least possible interference with the direct fixation rail fastener anchor inserts. If an insert falls too close to a transverse stirrup, the rail fastener spacing can be easily adjusted to remove this interference. However, if the stirrups are placed longitudinally and conflicts occur, they may need to be removed and reinstalled. That requires drilling into the invert slab and the potential for hitting embedded reinforcing steel is high. Also, since the conflict is typically not identified until plinth construction is well under way, the corrective action delays the overall work. The simple placement of the stirrups in the traverse orientation can therefore save a great deal of re-work and time.

13.3.2.4.2 Rail/Plate Support System

The objective of a support system is to set the two rails of the track to the exact gauge, alignment, surface, cant, and cross level prior to placing the plinth concrete. Development of a support system for the suspended rails and/or fastener plates is potentially the most important task of the project. It will certainly set the pace of construction and the quality of the finished product. If the wrong system is chosen, re-work is necessary, including possible demolition of new concrete. There will be many crafts involved in the construction of the plinths and the subsequent installation of the rail fasteners and rails, and representatives from each should be involved in

devising the support system. Including them will bring out the problems that may be encountered so that solutions can be made prior to the construction.

If an actual section of rail is used as a template, it is very easy to set these parameters. The first decision to be made is whether the permanent CWR will be used as the guide or separate template rails will be used. It is important to understand that when working in direct sun light, the rail will expand and contract. If 1400-foot [about 427-meter] CWR strings are used to support the rail fasteners in position, it must be understood that every 5 degrees F [3 degrees C] of temperature change will produce a ½ inch [12 mm] of rail movement. If the inserts are attached to the fasteners, and the fasteners are firmly attached to the rail, the expanding rail will drag the inserts through the wet concrete, causing voids on one side of the inserts. In contrast, if 300-foot [91-meter] rails that are anchored at their mid-points are instead used as construction templates, the extreme movement would be only $^{1}/_{16}$ inch [1 mm]. With this factor in mind, contractors have devised several approaches. These include gantry type systems. Others have used special rail clips that release the rail fasteners from the rail once the concrete has taken an initial set but before the rail temperature changes by any significant amount.

Once the rail is set, everything that is attached to the rail is automatically set in the proper orientation also. These template rails can be used over and over.

13.3.2.4.3 Handling Material

This is another decision that must be made early. Does the material get distributed in advance of the plinth placement or follow after the concrete is placed? If the actual direct fixation rail fasteners are not used as the holders of the inserts, they may just be in the way when concrete is being placed. If in a tunnel environment, temporary or permanent hatches to street level may be required in the roof structure, or hy-rail equipment may be the only way to distribute the material. Tunnels that include sharp curves can be a particular challenge for movement of materials. The constructor should be required to address these issues early so continuity of construction activities is achieved and material is not destroyed by other activities. Vandalism and theft may need to be brought into the discussion also.

13.3.2.4.4 CWR

Will the CWR be placed in advance of the plinth placement or after the plinth placement? If laying out the CWR in advance, certain considerations must be thought out. The CWR may buckle unexpectedly and injure a construction worker. It may simply get in the way of other activities and be moved aside by another contractor without the proper tools or experience. Other trades frequently do not understand that even a “slight” nick in the rail can be a serious defect. On one project, a surveyor placed cross cut chisel marks in the head of the rail for reference points. He had no idea that such chisel marks are the first step in a time-honored method of cutting rail without a saw. He also didn’t understand that even well-constructed track is subject to measureable movement due to loading and thermal effects and hence makes a poor reference point for surveying.

After the plinth is poured, a piece of equipment will be required to get the CWR into the fasteners. This can be achieved with laborers, but certain safety precautions must be adhered to so as not to damage or flip the CWR. Pushing the rail on top of the fasteners after the plinth has been placed will require a hy-rail piece of equipment equipped with 360 degrees of motion. Any

longitudinal movement of the rail requires rollers be placed under the rail. Whichever method is chosen, it must be well thought out.

13.3.2.4.5 Some Recommendations on Packaging

The way in which materials are packaged can affect the continuity of the work and the safety of the workers. Packaging should always consider how the packaged materials will be handled, including the equipment that will be used. For example, different packaging might be required for forklift handling versus crane handling. If a decision is made to layout fasteners in piles every 50 feet [15 meters], pallets could be assembled with the correct number of fasteners to accommodate that length of track. This logic should carry through to all material, such as bags of clips or bolts and washers. However, the way material is packaged must also match the equipment doing the unloading and distributing. The constructor must match the equipment to the packaging and, if possible, coordinate with the suppliers to match the packaging to the construction and handling methods. This all should be identified in the construction work plan furnished by the constructor.

13.3.2.4.6 Welding Rail

The flash butt welding process is the most common to produce CWR. The CWR strings are then welded together after the de-stressing activity is completed by the field weld process. Determining whether to fabricate the CWR strings onsite or off-site is governed by several considerations, including whether the rail is procured by the owner or the constructor.

Off-site welding presents some challenges concerning delivery. It's obviously only possible if there is a rail connection between the welding site and the construction site, since CWR is delivered by special rail trains. An evaluation must be made with respect to loading of the LRT structures. The axle loads of a rail train designed for freight railroad service is much greater than LRT, possibly by a factor of three or more. Also, the rail train might be incompatible with the curve radii, gradients, and perhaps even the track gauge of the LRT line. Cost-effectiveness will play a role in this decision, bearing in mind that the rail train delivery cost is usually on a daily rate and includes not only the time in transit from the welding plant to the job site, but also portal-to-portal time from some home terminal plus the time while it is being loaded and unloaded.

Because of these factors, most transit projects set up a portable rail welding plant, usually somewhere along the project site or at a convenient location along some existing segment of the LRT system. Setting up a portable welding plant is easy to do as long as there is enough length to stockpile the string lengths chosen. Welding onsite gives the flexibility of placing piles of CWR in strategic areas for later distribution. The welding site(s) must be accessible for delivery of the stick rail in lengths of 78 to 82 feet [24 to 25 meters].

13.3.2.4.7 Surface Preparation

The concrete invert for the direct fixation track (i.e., bridge deck, tunnel floor, or slab on grade) must be clean and free from chemicals that could prevent a good bond between the invert and the plinth concrete. The invert should also be roughened ("scabbled") to achieve a good bond. High-pressure water blasting is frequently used. The use of epoxy bonding agents, which was previously popular, is not recommended since their coefficient of thermal expansion can differ appreciably from that of the concrete.

Dowels/stirrups projecting from the invert need to be straight and clean. If epoxy coated, their coatings should be checked and touched up as necessary before subsequent construction makes them difficult to access and inspect.

13.3.2.4.8 Setting the Track

Setting both rails to the correct orientation is the essence of "top-down" construction. This can be accomplished by using a support system that holds each rail in correct alignment, gauge, and cant. The supporting system must be sufficiently robust to withstand both casual loadings and more severe impacts from workers and the subsequent construction activities. An example is the mass of a concrete pumping line and the pressure exerted by concrete flowing out of the hose.

The supporting system should generally be external to the finished concrete. However, jigs are available that are supported on smooth tapered rods. The rods can be positioned within the plinth formwork, embedded in the pour, and then easily withdrawn from the green concrete. The resulting holes in the concrete must be patched correctly, especially in a region that experiences freeze/thaw cycles and preferably before they become filled with construction debris.

The supporting fixtures should be capable of making adjustments. If the cant of both rails is 40:1, that parameter can be fixed. If the rail cant is deliberately varied so as to help wheel set steering, adjustments can be built into the jigs to accomplish that. If the gauge is fixed and does not need adjusting, correct gauge can be built into the system. There should be a means of adjusting elevation on both rails as well as alignment referenced from centerline of track or line side.

Line, Grade, Gauge, Elevation, and Cant: Setting these distinct parameters prior to placing concrete will greatly facilitate the success of a project. If this is accomplished correctly, the need for subsequent adjustments for tolerances will be dramatically minimized.

Attach Fasteners and Inserts: Once the rails have been set to the proper orientation, either the fasteners or accurate anchor insert templates can be attached to the rail. Use of the rail fasteners ensures accurate placement and saves the step of later removing the templates to install the permanent rail fasteners. However, it also subjects the rail fasteners to possible damage ranging from concrete spillage up through more serious harm. The constructor's work plan should address this topic directly so as to avoid disputes. The remainder of this discussion presumes the use of the rail fasteners as opposed to templates, but the principles are the same.

If the fasteners are fabricated with the rail cant built in, the plane surface defined by the bottom of the fastener will be the same for opposite fasteners. A simple check for cant is to place a 6-foot [2-meter] straightedge along the bottom of opposite fasteners. If there is a space anywhere along the straightedge, the cant is incorrect. If the rail is to be canted, but the fasteners do not provide cant, there should be a consistent gap above the straightedge on the field side of the fastener. This geometry must be closely watched during construction as a quality control checklist item.

Once the fasteners are attached to the rail, the inserts will be attached to the fastener by using a temporary bolt snugly holding the insert tight against the underside of a steel shim plate placed between the fastener and the insert. This temporary steel shim should be at least ⅛ inch [3 mm] thick and 1 inch [25 mm] larger in each plan dimension than the footprint of the fastener. This steel shim (also known as a "slobber plate") provides a smooth flat bearing surface on the

finished plinth for the rail fastener. The steel shim also prevents concrete from entering the voids on the underside of the direct fixation rail fastener. Using a plastic shim for this purpose is not recommended because the flexible properties of the plastic and its likely deformation (especially when hot) means that the finished concrete surface will likely not be a flat plane—making excessive grout repairs necessary. Thin layers of grout are not likely to withstand the repeated impact loadings from the rail fasteners, so it is strongly recommended to avoid construction methods that may require them.

The fasteners should not be attached to the rails using the permanent spring clips since thermal movement of the rail could drag the fastener inserts through the concrete. Instead, temporary clamping devices that have a point load on the base of rail should be provided. The only function of these devices is to hold the fastener plate firmly to the rail base during plinth construction. They should be released as soon as possible after the concrete has achieved an initial set. Other templates can be designed to hold inserts that may be used for an emergency guard rail or other hardware. As long as the geometry is calculated correctly, the inserts will be in the proper orientation for installing other attachments. The reference must be the running rail.

Install Reinforcing Steel: Installing the reinforcing steel or "rebar" can be done either before or after the rail is set. It would be wise to do a risk analysis on both methods to achieve the safest result for the workers with the best assurance of quality. The important aspect of this activity is to not move the rail out of alignment and to avoid contact with any embedments. When performing "top down," it is relatively easy to achieve a 2-inch separation between inserts and rebar. The difficult part is ensuring that either

- The coating on epoxy-coated rebar remains intact and that all chips and cut ends of the reinforcing steel are properly patched. The rail anchor inserts must be fully coated also, with no chips or "holidays" in the epoxy.

- Black steel reinforcing is properly welded/bonded so as to result in an electrically continuous cage. All welds must be checked and the electrical continuity of the rebar cage or mat must be checked. All it takes is one piece of rebar that is not part of the mat to compromise the design, initiate corrosion, and possibly destroy the structure in less than 20 years. There may be test stations associated with this methodology, and they must be checked also. If construction joints or control joints are used in the concrete, continuity must be checked between reinforcing steel on opposite sides of the joints. The objective is that every single piece of rebar in the structure is electrically continuous.

Obviously, either approach requires meticulous attention to details, particularly since such methods are not commonly encountered in other types of reinforced concrete. In many cases, even experienced workers installing the reinforcing steel will be unfamiliar with these methods and may not understand the importance of doing the job strictly in accordance with the plans and specifications. Close inspection is therefore required to verify that epoxy coatings are intact or intersecting black steel bars are welded. Quality control checklists are invaluable during this phase of the construction.

Concrete Placement and Consolidation: This can be accomplished in many ways. Using a concrete pumping system is the preferred method, but, if access is easy, a crane and a concrete

bucket can be used. If there is access alongside the track, concrete can be placed directly from the concrete truck. Whichever placement method is chosen, the concrete must be vibrated properly. Typically, the concrete contractor will use an internal vibrator. Some key points to keep in mind:

- It is very important that the right vibrator head be used. Because the plinths are usually only 6 to 10 inches [15 to 25 cm] tall, a short vibrator head, one designed for use with shallow slabs, is preferable to the more common 12-inch [30-cm] head. The short head is preferred because a long vibrator head will need to be held at an angle to minimize concrete splashing and thereby will mix more air into the molten concrete. A long vibrator head is also more likely to possibly damage any epoxy-coated rebar.

- The vibrator must be used properly. It should be inserted only in a vertical orientation. It should not be dragged so as to move the concrete. Pushing the vibrator under the rail fastener or slobber plate will create air bubbles that become pockets in the surface of the concrete beneath the plates. The vibrator should be placed only alongside the fasteners until the concrete is observed flowing under the plate and out the other side. It should not be left in any area too long as that will cause air intrusion and possible segregation of the cement and aggregate.

- The person operating the vibrator must be trained in its proper use. Sometimes, this task is assigned to an inexperienced laborer who has little understanding of what is supposed to be done. A few minutes spent training this worker can save hours of repair work made necessary because too much air was introduced into the molten concrete.

It is advisable to have a surveyor on hand who will monitor the position of the track and determine whether the track structure is being dislodged during concrete placement.

Any concrete spillage onto the direct fixation rail fasteners or the rail should be cleaned up immediately as part of the concrete finishing activities. Spilled concrete could jeopardize both the electrical and mechanical properties of the direct fixation rail fasteners. While concrete on the rail would likely eventually flake off during LRT revenue service, the resulting dust and debris on the trackway would be a housekeeping problem the owner should not have to contend with.

When finishing the concrete, it is important to maintain a flat surface between the fasteners with the finish grade located at the bottom of the steel plate.

Remove and Reposition Support System: After the concrete has achieved enough strength to support the rail and fasteners attached, the supporting fixtures can be removed and repositioned for the next section of track to be placed. If temporary template rails were used, their relocation to the next segment may need to be deferred until the concrete has achieved additional strength to accept loads from the rail-handling equipment.

It is important to evaluate the loads and strength of the concrete prior to removing the template support system. If CWR is used as the template rail, the concrete strength needs to be sufficient to accept the thermal forces from the rail. If shorter template rails were used, 16 hours is often

sufficient initial cure time, assuming proper concrete placement and curing procedures were incorporated. In colder weather, this may need some adjustment.

If the project is being constructed in an active revenue service line (such as the installation of a new turnout in existing direct fixation track), there will usually be a need to complete the work and restore the track to service as quickly as possible. Some projects have done this using high-early strength concrete with cure times as short as 20 hours.

Patch Voids under Fasteners: After the concrete has cured, the temporary anchor bolts will be removed so the slobber plate can be released and replaced with permanent shims (often polyethylene) as may be necessary to achieve exact final track profile. The slobber plates are removed, cleaned, and carried ahead for the next pour. The direct fixation rail fasteners should be thoroughly cleaned at this time to remove any remaining concrete spillage.

It is usually necessary to patch the small voids or air bubbles that may have been produced under the slobber plate during concrete placement. If trapped air was under the slobber plate, small voids (usually about $^{1}/_{16}$ inch [2 mm] deep and usually no bigger than 1 inch [25 mm] in diameter) will be observed. If this is the case, a cementitious grout product can be used to simply spackle this area. Once the grout has cured, the rail fastener can be reinstalled with shims as necessary. Larger voids, particularly any "honeycomb," could be indicative of improper concrete handling, should be examined by a structural engineer, and may require that the defective concrete be removed and replaced.

If temporary templates were used instead of the actual direct fixation rail fasteners, they will be removed and the permanent rail fasteners installed. If an automatic bolt centering serrated washer was used during concrete placement, this is an easy task. It is very important to index the entire system so the direct fixation rail fastener is installed or reinstalled in the correct location as either it or the template was positioned during placement of concrete. By indexing the plate with a zero adjustment serrated washer, it should be unnecessary to realign or regauge the rails.

TCRP Project D-7, Task 11, has addressed direct fixation track in extensive detail. See *TCRP Report 71*, *Volume 6*, for more details on bearing area required for the safe support of the rail and fastener.

Set CWR: If temporary template rails were used during plinth construction, the permanent CWR will need to be installed. In some cases, it may have been previously staged alongside of the track and will need to be threaded into position from the side. Other circumstances might dictate that it be brought in from the end of the segment. The decision will largely depend on the characteristics of the site, will have been made early in the project, and identified in the constructor's work plan. The rail will be seated onto the fasteners and temporarily attached to every fourth or fifth rail fastener using zero toe load clips. This will allow for any rail-mounted construction equipment such as hy-rail trucks or rail trains. The details on the temporary rail clips should be part of the construction plan developed by the constructor.

Even in the unlikely event that the CWR is set into the rail seats at the correct neutral temperature, full anchorage of the rail should not occur until after the rail fasteners have been

shimmed to provide the correct rail profile within tolerance and all rail fastener anchor bolts have been properly torqued.

Thermal Adjustment: Matching the concrete and rail stresses is an important part of this activity. Thermal adjustment of CWR in direct fixation track is similar to that described above for ballasted track with the following exceptions and cautions:

- On an elevated guideway, the guideway must be at a neutral temperature also. If the CWR is heated or stretched too much on a cold guideway, the rail, when it cools and contracts, will introduce forces on the guideway superstructure. These forces could deform the bridge's support bearings, especially if they are seismic bearings. It is advisable to defer rail thermal adjustment until the temperature of the guideway is close to its natural temperature when the rail is in its neutral state. Concrete and steel have similar coefficients of expansion and this must be taken into consideration when de-stressing rail on guideways.

- De-stressing rail in a tunnel is not a problem since the temperature ranges are not great. Note that the rail neutral temperature in tunnels will be significantly lower than for track exposed to sunlight. The range of temperatures experienced by rails in tunnels is also often higher on the low end since tunnels often stay warmer than the outdoors.

- The process of de-stressing and anchoring direct fixation track at grade is similar to work in tunnels except that the anchoring temperature will differ.

13.3.2.5 Drill and Epoxy Method of Direct Fixation Track Construction

This article is not intended to be a procedure for the constructor to follow. It is meant to inform the designer of certain activities that should be followed. The constructor should present their procedure to the designer prior to construction commencement and the designer should be prepared to present "what-if" scenarios that could result in unacceptable quality. Those scenarios should always be resolved before any construction begins.

13.3.2.5.1 Surface Preparation

Surface preparation of the base concrete slab for grout pads is generally as discussed in Article 13.3.2.4.7 with the exception that scabbling is limited to the footprints of the individual grout pads.

13.3.2.5.2 Scabble Concrete

The invert concrete should be roughened and cleaned prior to grout placement.

13.3.2.5.3 Survey for Grout Pads

Each individual grout pad must be identified and surveyed to a maximum tolerance of ⅛ inch. Identifying the footprint is one aspect. Identifying the elevation of all four corners is another aspect, especially for curved track on a gradient. Each pad will have a slightly different elevation and pitch in two planes. The superelevation must be laid out considering all four corners of each grout pad. The cant in the rail must also be surveyed if using zero canted fasteners. If canted fasteners are used, the grout pads opposite each other must be in the same plane. The formwork must be placed with all the parameters in mind. It would be advisable to have the constructor establish a detailed work plan to ensure that all the proper controls are in place. If the

pitch on the grout pad is incorrect, it will affect the cant of the rail and will generate an improper wheel-to-rail interface.

13.3.2.5.4 Pour Grout Pads to Rail Tolerances

The grout pads must now be poured. If a grout pad is greater than 2 inches thick, it may require some wire mesh. If wire mesh is used, care must be taken so it does not interfere with the holes that must be drilled later. Grout should be mixed and placed according to manufacturers recommended practice. A steel trowel finish should be employed. A flat surface is extremely important since the fastener must not wobble on the rail seat. This would cause breaking or cracking of either the fastener or grout pad under dynamic loading.

13.3.2.5.5 Layout Hole Pattern

The holes must now be drilled to accept the rail fastener anchor inserts or threaded rods. Drilling the holes could be done by core-drilling machines or pneumatic impact drills. Whichever method is used, care must be taken not to chip or break the grout pad. The hole pattern must be identified and the location for drilling marked. Care must be taken to drill holes perpendicular to the surface of the grout pad. Care must be taken on superelevated track. The holes will be plumb only when using canted fasteners on tangent track. All other conditions will require an angle other than plumb. Using zero cant fasteners will require a 40:1 batter inward from the vertical plane on tangent track. This geometry must be carefully thought out so problems do not occur later when the rail is installed. When laying out the inserts and measuring for gauge, the rail cant must be considered since track gauge is measured at the gauge line on the head of the rail, which will be at least 8 inches [240 mm] and sometimes even more above the plane of the grout pad. The size of the rail and the thickness of the rail fastener assembly must be known since the elevation of the gauge line of the rail will change the anchor insert layout geometry.

13.3.2.5.6 Drill Holes into Invert

Using templates that are specially designed and fabricated to match the drilling equipment rather than the actual direct fixation rail fasteners is recommended. The templates can be designed to make certain the holes are at right angles to the grout pad surface and incorporate stops so the hole is cored to the correct depth. The depth of the hole will typically be greater than the cover on top of the underlying reinforcing steel. Drilling through the top layer of reinforcing steel may be permitted, but a structural analysis should be performed to ensure that the integrity of the invert is not compromised. Also, if reinforcing steel is encountered, another analysis with respect to potential stray current must be performed.

Whatever grout or epoxy is used to install the anchor inserts, it should have good dielectric properties. The manufacturer of the grout or epoxy must be consulted for proper placement. Some require a smooth hole; some require the hole to have roughened edges. They all will require a clean dry hole prior to placing the epoxy or grout. Following proper procedures and quality control is essential.

Extreme care must be taken when drilling holes in any guideway. Some elevated structures using segmental construction methods have transverse and longitudinal pretensioning and postensioning strands. Under no circumstances should these strands be compromised.

13.3.2.5.7 Repair Grout Pads

After holes are drilled into the grout pad and the invert, some pads may be damaged or broken. These must be repaired properly in order to maintain a proper bearing surface for the fastener.

13.3.2.5.8 Clean Holes

Cleaning the holes is an important step in the process. If the holes are not properly prepared, separation may occur between the core-drilled hole and the epoxy surrounding the insert (or threaded rod) during the uplift cycle of dynamic forces imposed by the rolling vehicles.

13.3.2.5.9 Mix Epoxy and Install Anchor Inserts/Bolts

Mixing the epoxy or grout will depend on the manufacturer chosen. A procedure should be submitted and approved prior to any placement. The anchor bolts or threaded rods must be held firmly in the correct orientation when the grout or epoxy is placed. The proper batter, alignment, and elevation are critical since this activity now sets the gauge, alignment, and surface of the finished track. In most cases, epoxy is placed into the hole and the insert or threaded rod is "stuck" into the hole. Using the proper amount of grout is important so as to fill the hole yet not overflow it. Overflow would cause an elevated area bearing on the bottom of the fastener and cause it to wobble.

Alternatively, the inserts can be set first and then the grout pads poured. The top of insert sets the elevation for the pad, the inserts come out level with the pad, and the pad doesn't have to be drilled. Not drilling the pad can spare it the impacts associated with drilling, which can cause cracking. However, since the grout pad would be thinner than the insert is tall, it is still necessary to mobilize core-drilling equipment. Other work sequences are also possible, and it's very likely that grout pad type direct fixation track has been built in at least as many different ways as there have been contractors doing such work.

13.3.2.5.10 Attaching Fasteners to Grout Pads and Setting CWR

The fasteners can now be placed onto the grout pad and the anchor bolts loosely engaged. Setting the CWR on top of the fastener is done as previously discussed for top-down methodology. The rail can be attached to the fastener and the fastener attached to the grout pad. Once this is accomplished, elevation, cross level, and rail cant must be verified. If adjusting is required, shims must be added or the grout must be ground to the correct elevation to bring the vertical profiles within tolerance. All measurements should be recorded for each fastener and a cut sheet produced. This can be easily verified if the information is tabulated in a spreadsheet and a graph is produced with limit lines. Once the out-of-tolerance locations are identified, corrective measures must take place.

13.3.2.5.11 Shim Rails to Elevation or Grinding the Grout Pads

There are different materials that have been used for shimming purposes. Those are steel, plastic, high-density poly, stainless steel, and galvanized steel. When using a polymer base material, confirm that ultraviolet light will not break down the material. When using steel, ensure that this will not compromise the stray current protection. Note that slotted shims, while easier to install, may work out of position under dynamic loading and time. A locking mechanism may be advisable when using slotted shims. In some cases, it may be necessary to grind the grout pad to achieve the tolerances for elevation, cant, and cross level.

Shimming and re-canting can change the horizontal alignment and track gauge. For this reason, it is advisable to correct the vertical parameters first. Only once the vertical corrective measures have been finished should final horizontal adjustments be made.

13.3.2.5.12 Line and Gauge Rails

The line rail is always set first to the proper alignment using centerline of track indicators or a surveyor's transit. There is only so much adjustment in the slots of the fasteners, and one misplaced pair of inserts could affect several fasteners on either side. If there is not enough room in the slot to set the alignment, the inserts or threaded rods must be cored out and a new set epoxied in. When doing this correction, always maintain and account for the shims that may have been placed in the previous step. Once the line rail is brought to proper alignment, the other rail can be set to proper gauge. If there is not enough adjustment remaining in the fastener body, they must be drilled out also and reset.

13.3.2.5.13 Thermal Adjustment

After all aligning and gauging is completed and all bolts torqued, the thermal adjusting of the CWR can take place as previously discussed.

13.3.2.6 Embedding Inserts into Precast Segments

There have been some attempts to embed inserts into the guideway in order to fasten the direct fixation fasteners without either plinths or grout pads. The results have varied widely from reasonably good to disastrous. In the absence of extremely strict quality control, these methods can result in incorrect cant on the rail and bad horizontal geometry even after using up all of the adjustment capacity on the rail fasteners. In order to produce a safe reliable system, the gauge, rail cant, vertical alignment, horizontal alignment, and superelevation must be set within very tight tolerances. If the owner inherits a poorly constructed system, the cost of maintenance will be higher and the owner's performance, as perceived by the riding public, will be inferior.

The advantages of construction without plinths or grout pads include the following:

- If done correctly, it can save track construction time since many of the tedious procedures described above for both plinth and grout pad direct fixation track can be omitted. However, significant additional time is necessary at the casting yard to be certain the anchor inserts are in precisely the correct locations.

- The structural dead load will be somewhat less due to the omission of the plinths.

Factors weighing against this type of construction include the following:

- Absolute cooperation and extremely close coordination is required between the contractor(s) responsible for erection of the aerial structures and the contractor responsible for the track.

- The structural engineers cannot control the camber of the finished bridge deck to the extremely tight tolerances specified for track construction. The result may be far more shims beneath the rail fasteners than might be prudent. This could result in high moment loadings of the rail fastener anchor bolts. The anchor bolts might need to be provided in

a variety of lengths so as to provide a reasonably consistent amount of thread engagement. Verification that the appropriate length bolt has been used at a particular location would be very difficult and tedious.

- Reduced clearance between the base of the rail and the invert. If this distance is less than about 2 inches [5 cm], debris such as leaves, blowing newspapers, and other paper trash may get caught under the rail. Those items in turn will catch and hold smaller debris. In addition to blocking drainage, this could ground the track system, causing problems with the train control system as well as providing a path for stray current.

- Any discrepancies in the as-cast positions of the inserts will not be apparent until the rails are set and the contractor is attempting to set final alignment and gauge. Any corrections for horizontal alignment may require drilling holes in the post-tensioned segments. This will carry a very high risk number.

Prior to any decision to cast inserts into the segments, it would be prudent to do a full and complete risk analysis to ensure that risk is mitigated to a level acceptable to all parties—the constructors, the designers, and the owner. If this method is chosen, it is essential to think everything through in extreme detail. One mistake in this methodology can produce an inferior product that will fail prematurely.

13.3.2.7 Top-Down Methodology Advantages

Since “top down” is the preferred method of most track constructors, this discussion will describe the principal advantages of this form of construction. If the track system is built properly, an owner can have a system that will be safe and reliable and will last for a very long time. All of the items are achievable with good quality control and an experienced track constructor. The following advantages are noted:

- Since all metal components are placed prior to the placement of the concrete, visual inspection will generally ensure that proper isolation (or bonding) has been achieved.
- Since the rail is set first, any type of fastener—flat or canted—can be used to achieve any desired amount of rail cant. This can include zones of transitional rail cant such as from the standard 40:1 up to a 20:1 for improved curving or down to a zero cant in a turnout.
- It is possible to use rail fasteners that do not incorporate lateral adjustment capability since all adjustments can be made in the jigs prior to concrete placement. This reduces the cost of the rail fasteners themselves. Several owners eschew lateral adjustment capability in their rail fasteners on the grounds that they never need it for maintenance and including it merely becomes an excuse for the constructor to be lax on quality.
- No relining, re-gauging or re-setting elevation. Once the rail has been set the first time, there is no need to set these parameters again assuming the constructor took the proper precautions and used the proper templates.
- Since the rail is set first, the proper cant is guaranteed.
- Rebar can be inspected prior to placement of concrete, and, since there should be no drilling into the concrete, the integrity of the reinforcing steel is maintained.

- It is always more cost-effective to do it right the first time. Re-work is costly and will affect both the quality of the track structure and the money aspect to achieve the tolerances.
- Top-down methodology creates the best end product and one that is not patched and scarred because of re-work. It is absolutely possible to produce a quality project that will last for decades.

It is the designers' collective responsibility to educate themselves on the likely construction methods and particularly to research the problems that have happened on previous projects due to ill-conceived construction methods, poor quality of work, and poor workmanship. An impeccable quality control program will be the guide to success. It must be adhered to and personnel held accountable for both success and failure.

13.3.2.8 Lessons Learned the Hard Way

There have been some very important lessons learned during both the design and construction phases of light rail systems. It can be a valuable tool to understand during the design phase of a project. We have an obligation to learn from our mistakes and to share them with the industry. The ultimate user of these systems must be safe, and the designer and constructor have the responsibility to deliver this to the owner. Some of these lessons learned include the following:

- **Getting the rail cant wrong:** If the constructor has not set the rail cant properly, it will make things very difficult for the Maintainers of the system. It will cause poor wheel/rail interaction, which leads to poor ride quality, truck hunting, accelerated wear on both the wheels and rail and possibly even rail defects. Some properties have ground the rail head to match the 40:1 cant that should have been produced by the rail fastening. However, this causes the force vector (L/V) to move outside the web of the rail, causing unnecessary forces in the head-to-web area of the rail section. Using any method other than "top down" will have a much greater risk of causing this poor relationship between wheel and rail.

- **Voids under the plates:** Small voids or air pockets are common to placing concrete under a flat surface. However, as long as these are not large voids due to improper concrete consolidation, they will not affect the integrity of the structure and can be easily patched. They can be minimized by use of a concrete mix design with a low water/cement ratio and through proper concrete vibration methods. Using the concrete vibrator correctly will greatly reduce the volume of trapped air.

- **Crooked inserts:** Inserts that are not perpendicular to the bottom plate of the fastener can be at risk to shear since the lateral forces are concentrated on a very small area. Crooked inserts are caused by poor quality control when performing a checklist prior to placing concrete. Each insert should be touched to ensure it is snug to the base of the steel shim. Just because it looks vertical does not mean it will not be moved by the flowing concrete.

- **Rail expansion during concrete pours:** If the rail length changes before the concrete has taken an initial set, there is a good chance the rail will drag the rail fasteners and the anchor inserts through the molten concrete. Understanding the forces that can occur due to temperature change will allow for proper procedures that can substantially reduce the

risk. Rail temperatures rise dramatically during the morning and then level off for many hours before finally decreasing after sunset. Either a consistently sunny day or a consistently cloudy day is perfect for concrete placement. Placing concrete at night when temperatures are relatively constant can also offer relief from this factor. Using lengths of rail no longer than 300 feet [91 meters] can reduce the risk. Using temporary rail clips that exert a very low toe load will mitigate the risk. Some temporary clips include a thumbscrew arrangement so the clamping force can easily be reduced as soon as the concrete has taken an initial set. Good planning and understanding of the forces involved will greatly reduce this risk factor. Identifying rail temperature ranges for concrete placement will reduce the risk. Performing a rail temperature analysis to determine the best time to place concrete will save many problems later.

It is a good practice for the engineer to consult with others that have performed, designed, and maintained direct fixation track structures. They will gain valuable information in order to make intelligent decisions based on lessons learned. Each project has a set of circumstances that may be slightly different than another project, but in general they have many similarities in means and methods as well as lessons learned.

13.3.3 Construction Issues for Embedded Track

The purpose of this discussion is to offer procedures for the construction of the embedded trackform. There have been many methods attempted, and some have resulted in failures. Unfortunately, the failures may not be recognized until years after their construction. This article will describe not only some proven methods but also some of the less advantageous procedures that may require a higher level of track maintenance.

13.3.3.1 Construction Concept

Understanding the different methods of construction may help the designer when choices must be made with respect to material, location, and geometry. Different methodologies offer varied advantages and disadvantages. The same philosophy previously recommended for direct fixation track will apply here with the exception that the concrete is placed to the top of the rail, fully encasing the rail in concrete. This presents some challenging problems such as how to deal with noise, vibration, and electrical isolation, as well as a proper running surface for motor vehicles.

13.3.3.1.1 Top Down

This is a widely used methodology and has been accepted as “best practice” and hence become an industry standard. Placing concrete after the rail has been set and tested for electrical isolation is the preferred method. With proper quality control during construction, this process results in a track system that should last for decades with minimal maintenance.

13.3.3.1.2 Bottom Up

Placing the concrete first and only then adding track components until the top of rail is achieved relies totally on placing concrete to within the tolerances required of the track. It is far more difficult to place concrete within ⅛ inch [3 mm] than it is to set the rail to that tolerance. When designing these systems, the construction methodology must be taken into consideration.

13.3.3.1.3 Drill and Epoxy

When drilling holes in concrete and placing inserts or threaded rods, it must also be performed within the track tolerances. In some cases, these tolerances have been set to $^{1}/_{16}$ inch [1.5 mm]; however, in most cases the tolerances are ⅛ inch [3 mm] .

13.3.3.1.4 Slab Concept

This concept places a flat concrete slab and then builds the track on top of that. This offers the constructor the option to use either a top-down or bottom-up method.

13.3.3.1.5 Tub Concept

If this method is chosen, the subgrade must be placed within the tolerances of the track when precast tub sections are used. This is another form of bottom up. Extra care must be taken when preparing the subsurface. The subsurface must be well compacted so no settlement occurs later, during rail installations.

13.3.3.1.6 Trough Concept

This concept places the concrete first, leaving two "slots" or troughs in the slab for the rails to be set. In some cases, the threaded rods are installed during concrete placement. This requires extreme care when choosing the formwork to hold the rods in place during placement of the concrete. The formwork for the troughs must be well supported in order to accept the pressure of the concrete, especially uplifting forces that might cause the formwork to "float." The elevation of the concrete pavement will need to be placed very close to the tolerances specified for the track, which will generally be much tighter than the concrete contractor is used to.

The geometry must be well thought out when mixing roadway with railway, and extra care must be taken when placing concrete on a grade where the track curves within the gradient. Roadway surfaces are measured perpendicular to the alignment of the roadway, and track elevations are measured perpendicular to the rails. If the track profile and the pavement grades have not been very carefully coordinated, this can result in a rail appreciably lower or higher than the adjacent concrete surface. If the rail is left above or below the roadway surface, serious accidents can occur (and have occurred) involving roadway vehicles, especially if the roadway and railway alignments are horizontally skewed to each other.

13.3.3.2 Preparatory Work

Because embedded track construction generally occurs within public rights-of-way and on constrained worksites, it is extremely important to "plan the work and work the plan." The following activities can help a project "get off on the right foot." Preplanning and communication will only help the execution. Many of these categories can be "show stoppers" to the construction, which will only lead to claims, change orders, and possible litigation. Possible problems should be identified and, to the degree possible, mitigated prior to construction. Preparatory factors that should be investigated include the following:

- Providing a site plan showing utilities and other pertinent data. This will build upon the construction contract drawings, adding information that is particularly relevant from a constructor's point of view and that is reflective of the constructor's means and methods.

- Determining accessibility, that is, for each discrete activity, how the constructors will get materials, equipment, and labor to and from the worksite.
- Identifying the volume of vehicular traffic for the purpose of road closures and detours.
- Identifying pedestrian traffic paths and volume and determining how to prevent harm to the general public.
- Conducting a pre-inspection of the area to account for and mitigate any potential problems.
- Performing a baseline stray current report prior to any construction activity to at least verify that levels are acceptable before construction begins.
- Investigating the geometry of the roadway surface and superimposing the geometry of the track to identify locations where the two may not be synchronized. Locations of deliberate reverse superelevation should be clearly identified.
- Confirming that hazardous material, permitting, and detours have been thought out.
- Developing and reviewing plans and procedures for disposal of excavated material.
- Considering fire and emergency response during construction.
- Recognizing and planning around “Dig Safe” and other early warning notifications prior to construction activities.
- Performing an overall risk analysis and survey.
- Performing regular document control, scheduling, and planning. Proper and timely audits should be the norm to ensure correct controls are in place and being performed.

This is a minimal checklist, and there may be other items specific to the project that need to be added. Thinking through the project in great detail while playing the “what-if” game can save countless hours of frustration and discontent during the course of any project.

13.3.3.3 General Overview of Construction

This article explores the construction activities that will be required to build a safe and reliable system. The intent of this article is to offer the designer some “food for thought” while designing the system. If there are different choices for design, offering one that is easier and more cost-effective may be the owner’s preference. Understanding the construction process can only help in the design decision-making process.

13.3.3.3.1 Excavation and Drainage

Once all the preparatory work is completed and any high-risk items mitigated, the construction phase can begin. It starts with a saw cut in the existing pavement and excavating to the correct depth. A drainage system would be installed at this time and is generally in the form of an underdrain consisting of perforated pipes with the holes facing down surrounded by crushed

stone and wrapped in a filter fabric. Cleanouts are typically placed at strategic locations, mainly in the bottom of vertical curves.

13.3.3.3.2 Compaction

This should never be overlooked. The subgrade becomes the foundation for the highway and railway. Many properties will use an asphalt underlayment of 6 to 8 inches [15 to 20 cm] thick to guarantee 100% compaction. Evaluating the additional cost compared to the potential problems is essential. In well-compacted soils, this may not be necessary. Where organic material or soft soil is encountered, this may be the norm. It may be necessary to overexcavate to remove this unwanted material. If this is the case, new fill will be needed. This fill material should be suitable and compatible with the adjacent soils so as not to permit uneven or abrupt settlement. The engineer must realize that this activity must be scrutinized and ensure that good quality controls are implemented.

13.3.3.3.3 Installing Underground Electrical Conduits

Installing the electrical conduits can be very challenging since in some cases the signal and other electrical systems may not yet be fully designed. Adding a few extra conduits in strategic areas can save countless hours of chopping concrete and destroying the integrity of the embedded track. The signal system may need impedance bonds, and block outs in the concrete may be necessary. Negative return systems will need wires also as well as the grounding system, depending on whether collection or isolation is chosen. Engaging the electrical constructor during this activity is very important. Proper coordination among all parties involved should be standard procedure.

13.3.3.3.4 CWR

Much like with direct fixation track, decisions must be made as to how the CWR will be handled and placed. Unlike the direct fixation method where the rail is exposed, the actual rail must be used in the construction since it will be fully encased in concrete. String lengths may be governed by the amount and length of road closures that will be permitted. Intersections are always a challenge and must be well thought out. Will the intersection be available for a full closure or must it be completed one half at a time? Will temporary roadways be required or detours implemented? These and other questions must be answered to determine the proper lengths of CWR to be used.

13.3.3.3.5 Construction in the Urban Environment

Working in an urban street environment is not the same as working in a fenced in building site or a rural area. Extra care must be taken to protect the general public. The general public typically understands very little about the construction process and cannot be depended on to watch out for its own safety. Detouring people and vehicles is the safest approach. If appropriate measures have been taken to either exclude the general public from the construction zone or to carefully manage and control its interaction with the construction, it reduces the chances of “incidents” and risk analysis numbers are low. Fences and barricades can prevent exposure to dangerous activities and material. Rerouting sidewalks will reduce the possibility of a pedestrian wandering into the construction site and getting hurt. In many cases, management of pedestrians is more difficult than motor vehicles, particularly in cities where “jaywalking” is common behavior.

13.3.3.3.6 Safety

Safety is the number one concern of all parties involved including the general public. A professional, well-trained safety engineer should be employed and always invited to meetings offering a perspective from a safety point of view.

13.3.3.3.7 Welding Rail

When welding rails into strings, it is necessary to make some decisions as was done for other trackforms. In most cases, it may be advisable to simply weld long strings and cut from each string the lengths that are required. Close controls are required to ensure that each piece of each string has a designated location and the location is documented along with the heat numbers associated with that piece. Care must also be taken to confirm that head-hardened rail is placed properly and according to the specifications. It is common practice to install head-hardened rail in sharper curves since this type of rail can offer 40% more life.

The placement of field welds is critical for this form of track. Criteria are included in the specification on where field welds should not be placed, such as at road crossings, within 15 feet [4.5 meters] of another field weld in the same rail, or within 5 feet [1.5 meters] of another weld in an opposite rail. These criteria would be identified by the designer or the owner.

13.3.3.3.8 Setting the Track with Isolation

Setting both rails to the proper geometry is similar to direct fixation track construction methods; however, supporting fixtures, such as I beams or plastic ties, can be encased in the concrete and used as sacrificial elements to hold both rails in the proper geometry. Applying a rubber boot or other isolating membrane is critical to the longevity of the system. When using a rubber boot or other method, it must be checked for imperfections in order to confirm that the rail is isolated. This can be accomplished by using a megger type system that discharges an electrical current that is continually seeking ground. If there is a pin hole in the boot, the testing system will create a spark indicating that the membrane has been compromised. This hole must be properly patched per manufacturer-recommended practices. Ignoring this final check prior to placing concrete could cause serious repercussions later and will most assuredly produce a failure mode when track-to-earth resistance testing is performed. This isolation system should also serve as a form of noise and vibration protection. Concrete placed directly in contact with the rail will fail in a short period of time since concrete is not designed to withstand the vibration imposed by trains. It would be in the engineers' best interest to explore all the possibilities and educate themselves about lessons learned with respect to this topic. The engineer always has the ability to write a performance specification instead of identifying specific products. This is a decision that the owner and the designer make together.

13.3.3.3.9 Thermal Adjustment of Embedded Rail—Required?

In ordinary ballasted track, it is essential to adjust the rail to a zero thermal stress temperature so as to minimize the chances of track buckling during hot weather and pull aparts during cold weather. Track buckling is extremely unlikely to occur in embedded track because the mass of the surrounding concrete pavement will both constrain the rail from lateral movement and act as a heat sink, keeping the rail temperature well below that which might be experienced in an open trackform. Doubtless because of these factors, experience has shown that it is unnecessary to heat embedded rail during laying. Instead, it can simply be encased in the concrete at reasonable temperatures. The rail will assume the temperature of the concrete as the latter is

placed and cures. Notably, the concrete pour itself will have temperature range restrictions (typically 40° to 95° F / 4° to 35° C) based on Portland Cement Association recommendations. In addition, for embedded track that uses rail boot, slippage between the boot and the rail will generally relieve the small amount of thermal stress that can occur as the rail matches the temperature of the curing concrete. Hence, it is unnecessary to de-stress embedded rail so long as it is in a zero-stress condition at the time of the concrete pour.

The greater temperature-related issue in top-down embedded track construction is keeping the skeletonized track in alignment prior to and during the concrete pour. As is the case with top-down construction of direct fixation track, the position of the rails should be closely monitored during the pour.

13.3.3.3.10 Placing Initial Concrete Only up to Base of Rail—or Not

Once the rail has been set to the proper grade, line, and gauge, it is time to pour the concrete. A decision must be made as to whether to place the concrete to the top of rail in one lift (single-pour method) or stop at the base of rail and make a second concrete pour on top of that (two-pour method). If a topping other than concrete, such as brick or blockstone pavers is preferred, the two-pour method is required. In some instances, the top of the slab will be colored concrete stamped with a decorative design. This would require a two-pour methodology. Other issues to consider include the following:

- When placing a large depth of concrete, it is awkward for workers stepping in the wet concrete since the risk of a twisted ankle or other injury is higher.
- If the two-pour method is chosen, it gives one last opportunity to verify total electrical isolation of the rail when repairs can still be made without chopping concrete.
- A two-pour system introduces a cold joint between layers of concrete, so the first pour must remain roughened and no curing compound should be used.

The decision of whether to use a single- and two-pour method is often best left to the constructor, since some are better than others at concrete placement. Whichever method is chosen, the most important aspect is again good quality control and producing a checklist as verification.

13.3.3.4 Lessons Learned the Hard Way

There have been some very important lessons learned during both the design and construction phases of light rail systems. It can be a valuable tool to understand during the design phase of a project. We have an obligation to learn from our mistakes and to share them with the industry. The ultimate user of these systems must be safe, and the designer and constructor have the responsibility to deliver this to the owner. The bullet points below are some lessons learned from many constructors and designers:

- **Placing concrete below the top of rail:** When constructing embedded track that will be shared with rubber-tired vehicles, it is imperative that the top of the rail remain even with the top of the concrete except that an area within a short distance on the field side may be left depressed ⅛ to ¼ inch [3 to 6 mm] to account for hollow wheels and prevent the wheel from contacting the concrete. On one project, a section of rail that was skewed to the path of motor vehicles was sticking out above the concrete by almost ½ inch [13 mm]. This condition was a significant contributing factor to a fatal motorcycle accident.

- **Not testing the boot or isolation method**: If the boot is not tested for holidays prior to placing concrete, the track-to-earth resistance test may fail. If this test fails, it is quite an ordeal to locate the problem. Once the problem is located, concrete must be removed and the insulating membrane repaired. Then new concrete must be placed to repair the hole. This vertical joint in the concrete is very likely to become a maintenance headache due to freeze/thaw cycles or water migration.

- **Not testing rebar continuity**: The rebar mats should be tested for continuity if the collection method is chosen. If the mats are not continuous or the return path is not properly connected in the substation, the current will stray and begin damaging the steel components at a rate of 20 pounds [9 kg] per ampere per year, according to Faraday's law. If concentrated in a few locations, this corrosion can be very dramatic over a short period of time and may jeopardize the entire structure. Therefore, be cautious when choosing the collection method.

- **Not monitoring rebar and concrete placement**: When a crew places concrete, it may be thought of as organized chaos. Using checklists before, during, and after concrete placement can save money, time, and embarrassment. However, concrete pours for embedded track are distinctly different from ordinary roadway work. The typical roadway concrete pavement crew is used to working within a tolerance of ¼ to ½ inch [6 to 12 mm] and believe they are doing really well if they meet the lower end of that scale. However, the completed track usually must be within ⅛ inch [3 mm] of the specified alignment. Communicating this tight tolerance requirement to both the concrete superintendent and the actual concrete crew is critical since it is the actions of the latter that will possibly knock the track out of alignment.

  It is highly desirable to have a survey corps present to check and recheck track alignment immediately before, during, and after the concrete pour. The trackwork constructor should also have at least a supervisor present during the pour, and it should be understood in advance that any dislocation of the track will require a suspension of the pour until after the misalignment has been corrected. If an alignment defect is not discovered until after the concrete has set up, it will need to be chopped out to correct the defect. The demolition of the green concrete is virtually certain to damage not only the embedded rebar but also the rail boot. The cost of reconstruction would be substantial, making it highly desirable to "get it right the first time." The location of the rebar must be monitored also. If the concrete flow moves the rebar cage, it may damage the rail boot and result in a "hot spot" with a path for stray current to ground.

- **Inexperience**: A good track constructor makes track construction look very easy. It is tempting for the novice contractor to tackle these projects without understanding what is really involved. (This caveat applies to designers as well.) Inexperience can only be overcome by experience. Many public agencies are legally required to award contracts solely on low price; however, if it is permissible, owners should strongly consider pre-qualifying and shortlisting contractors prior to asking for bids or issuing a request for proposals. The experienced constructor who costs a little more will usually result in the best value to the owner. The end user of the light rail system depends upon and expects

a safe and quality product for themselves and their families to use. It is the responsibility of everyone on the project to do their best to achieve this.

- **Not following a QA/QC Program**: This has been mentioned throughout this chapter and has been emphasized in a very strong way. An impeccable QA/QC program should be a mandatory requirement. While having a complete and auditable paper trail is one aspect of a comprehensive quality program, actually following the procedures is the far more important part. Audits are a check to ensure proper implementation. Qualified people that have support from the highest authority are the secret to success. Build a safe, reliable system that everyone can enjoy for generations to come.

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Chapter 14—LRT Track and Trackway Maintenance

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CHAPTER 14—LRT TRACK AND TRACKWAY MAINTENANCE

14.1 PURPOSE

The designer should understand that once the system is designed, built, and turned over to the owner, it will need to be maintained. All too often, the focus of the designers and constructors is only on building the project with insufficient attention to the interests of the maintainers. In many cases, it has been assumed by the owner that maintenance will not be required. The fallacy of such assumptions is usually not apparent until conditions have deteriorated to a level where corrective measures are difficult and expensive and cause significant inconvenience for all parties, including the system users.

If a system of periodic inspections and preventative maintenance is in place from the beginning, all properly designed and constructed LRT trackforms should last for decades without requiring significant reconstruction. This is largely because of the small axle loads of rail transit vehicles.

The purpose of this chapter is to offer some recommendations for maintaining all trackforms so that the designer has a basic understanding of the requirements. Much of maintenance is generic to all trackforms, but some require more specific focus to achieve success. FRA, APTA, and the FTA have developed some general criteria that can be useful, but it is the responsibility of each owner to develop a maintenance program specific to its system.

It is important to include the owner's maintenance-of-way (often abbreviated as either M/W or MOW) department in design decisions that may affect maintenance after construction. The maintenance-of-way department may have existing standards and methods in place that can be directly applied to the new construction whereas new methods may not be applicable to their existing infrastructure. Keeping things simple for the maintainers goes a long way toward ensuring that the system will be maintained and remain in state of good repair. In the case of a new LRT system that does not yet have an in-house maintenance organization, the designer may be in the best position to initiate a comprehensive plan for inspection and maintenance of LRT track.

14.2 TRACKWORK STANDARDS

The designer must set certain tolerances for the construction entity to build the project. These standards are completely different than either maintenance standards or safety standards.

14.2.1 Track Construction Standards

Construction standards are tolerances that must be met during the building portion of the LRT system. See Chapters 4 and 13 for discussions on construction tolerances. Construction tolerances typically reflect the use of new, unworn materials and typically cannot be achieved once the system is in operation and the components have begun to wear.

14.2.2 Track Maintenance Standards

Maintenance standards are for maintainers to use and not for new construction. They represent desirable minimum limits so that satisfactory operations can continue. Maintenance standards, like the safety standards discussed in the next article, are typically established for certain speeds

and classes of track. These standards will be unique to each transit property based on its needs and objectives. Inherent in any maintenance standard is an expectation that a certain amount of deterioration will occur before the next cycle of maintenance but that ride quality will not be impaired to the degree that corrective action becomes mandatory.

Track maintenance standards and related requirements are typically codified in manuals and other documents developed specifically for each railway. These procedures will address many conditions and may include the following:

- Track charts of the property identifying speeds, alignment, curve radius, gauge, and cross level
- Criteria for drainage and vegetation control
- Maximum allowable dimensions for track gauge and alignment, both vertical and horizontal
- Criteria for maximum acceptable ballast and cross tie deterioration
- A CWR plan (sometimes called “track buckling countermeasures”)
- Criteria for inspection and maintenance of plinth concrete and embedded track slabs
- Criteria for inspection and maintenance of turnouts
- Criteria inspection and maintenance of other track appurtenances such as bumping posts, rail expansion joints, etc.
- Criteria concerning resuming operation after a severe weather event such as high water
- Torque limitations for anchor bolts and other threaded fasteners
- Inspection frequency and record-keeping requirements

The above is only a partial list, and reference should be made to standards and recommended practices in the industry. Many properties will assign color codes to various criteria to establish degree of severity and actions to be taken.

14.2.3 Track Safety Standards

Track safety standards are criteria that signify absolute limits for safe operation at various speeds. If the track structure deteriorates beyond these safety standards, then immediate action must be taken. That action might be

- Correction of the defect so as to bring the track into compliance.
- Reduction of train speed to a level consistent with the conditions that exist.
- Removal of the track from service until such time as the condition has been corrected.

Typically, at least two of the actions noted above will be required in response to any individual class defect. For example, if the defect cannot be corrected immediately upon detection, a speed restriction would be imposed (first action) until a crew can be mobilized to correct the problem (second action). Obviously, the preferred circumstance would be that the track never deteriorates to a level where safety limits become a consideration because maintenance standards have been adhered to.

While the Federal Railroad Administration has a comprehensive set of track safety standards (see 49CFR213) that are a useful reference, they technically do not apply to urban rail transit systems that are not connected to the general railroad system. Moreover, there is much in the FRA Track Safety Standards that could be misleading for a system that does not employ AAR wheel profiles and wheel gauges. APTA published safety standards for the transit industry in 2007. They are great resources for an agency to develop both maintenance standards and safety standards but are necessarily generic and therefore must be adapted to the specific conditions of each rail transit line. As of 2010, the FRA has been charged by Congress to develop safety standards for transit systems that are not connected to the general railway system. This will ultimately result in fines for noncompliance.

14.3 ACCEPTANCE

Once a track is accepted by the owner, their maintenance organization (which might be either an internal department or an outside contractor) will take over responsibility for the track from the constructor. The designer should understand what is important to the owner before the maintenance organization assumes responsibility. The following are some of the minimum requirements prior to transferring responsibility from the constructor to the maintainer:

- Documentation—the proper documents will need to be turned over to the owner upon completion of the project. These documents should have been gathered throughout the course of construction according to the QA/QC program. These documents are an integral part of the development of standards for maintenance. Not only will this pertain to the track but also to systems, vehicles, structures, and other pertinent parts that make the whole. It is common for maintenance manuals to be developed by the builder and turned over to the owner upon completion. Sometimes these documents are not produced because of the sense of urgency to start revenue service. Once revenue service begins, many items are forgotten, and sometimes the forgotten items include maintenance manuals. If proper audits and good quality control were maintained during the construction phase, this should not be the case.

- Final Inspection—the owner should take part in the final inspection and acceptance. Only after any discrepancies have been resolved should revenue service begin and the maintenance group take over. The maintenance group should not be left with the task of completing punch list items.
- Satisfying NCRs (Non-Conformance Reports)—any NCRs that were generated during the course of the construction must be signed off on and accepted by the engineer prior to acceptance and release of any retention money to the contractor. Refer to Chapter 13 for more detail about NCRs. A good QC program will track NCR disposition and put up red flags if not satisfied during a reasonable amount of time.

14.4 WORKING SAFELY ON THE RAILWAY

Safety is more than just important, it is essential. Safety must both precede and be concurrent with any actual maintenance work. Readers who are familiar with railroad engineering and maintenance organizations know how they invariably start any activity, even a meeting within an

office, with a safety briefing. The purpose of the briefing is to make certain that everybody understands the activity to be performed, the procedures for doing it in a safe and approved manner, and, in the event of an emergency, what to do to minimize injury or other harm. Following that example, this discussion on rail transit system maintenance will be prefaced with a discussion of safety.

Maintaining the light rail transit line is distinctly different from building it because many maintenance activities must be conducted during train operations. The approach toward safety therefore needs to be different as well. Roadway worker protection has been an important topic in the industry with recent changes and modifications. Understanding how the railway works and how to be safe when working on the railway are paramount to the safety of every person.

14.4.1 Track Protection

The track designer must understand how people will maintain the track and the personal safety procedures they will follow and then consider whether it may be necessary to include infrastructure or systems features that will facilitate their safety. Protection of on-track workers from approaching trains can take many forms depending on the configuration of the work zone, the speed of approaching trains, and the type of work being done. At one extreme is implementing temporary “trip stops” to absolutely stop a train if it enters a work zone. More common is a simple line-of-sight procedure using flaggers who warn of an approaching train, somewhat similar to how highway crews are protected during maintenance work. Determining which protection is appropriate for various types of work is the decision of the owner and will typically be codified in written procedures. Issues to be addressed by the procedures include questions such as whether the maintainers will be permitted to work on live track, and, if so, how they will be protected from being hit by a train. Training of the workers in the proper procedures to follow and the possible dire consequences of not abiding by the rules is paramount.

14.4.2 Flag Persons

Persons that observe train movement and are in contact with the dispatcher are commonly called flaggers. Their sole responsibility is to ensure that the work crew is safe from passing trains. They need a place to stand where they will have clear sight of both an approaching train and the work zone. They and the workers also need a place to clear up from a passing train. The engineer must give this some thought when designing a system in order to build in clearance areas and identify no-clearance zones. A simple niche in the wall could prevent someone from getting hit by a train.

14.4.3 FRA 214 Regulations (If Applicable)

FRA 49CFR214 lays out rules for the safety of “roadway workers” on any system connected to the general railroad system. Agencies subject to FRA oversight must adhere to 49CFR214, or fines can be imposed. While, strictly speaking, these rules may not be legally applicable to a given agency’s LRT operation, they do represent “good practice.” The track designer should therefore read this document as well as similar APTA-recommended standards to gain additional understanding and knowledge of how the system must be maintained. This should lead to the incorporation of system features that foster a safe working environment and facilitate maintenance activities.

14.4.4 Risk Analysis

Mitigating risk to a safe level will reduce accidents. Thinking about what could happen might just save someone's life. Having an action plan for every conceivable problem that could go wrong may get all the workers home safely to their families.

14.4.5 Preparatory Work

In some cases, a work zone can be viewed prior to actual maintenance being performed. This is a good idea that will help develop good work plans and allow the proper tools to be available during the work outage. For example, if a tie is to be changed and the workers arrive on the site to find a piece of loose rail in the way, they will need a tool (rail tongs) to get the loose rail out of the way that they may not have brought with them (because it is not necessary for changing a tie). Now a worker must leave the safe work zone and retrieve rail tongs. This wastes valuable track time as well as worker time. In order to prevent this, a good foreman would have inspected the work zone, possibly by riding a revenue train the previous day, and thereby discovered that rail tongs would be required for this task.

14.4.6 Profile of a Safe Person

What constitutes a safe person? If an individual answers positively to five or more, they are relatively safe. Less than five could be an unsafe person, and no positive responses are an accident waiting to happen. We only offer this as general knowledge.

Experienced, well trained	Feels good about self
Alert and clear headed	Neat and orderly
Not easily distracted	Able to follow direction
Familiar with equipment	Understands objectives
Aware of consequences if unsafe	Concerned for fellow workers

14.5 INSPECTING THE SYSTEM

Only now, after discussing safety above, can we let the reader "get out on the railroad." Much of this is identified in the APTA track maintenance standards, which are patterned after the Federal Railroad Administration's Track Safety Standards, also known as 49 CFR Part 213. The FRA document applies only to those agencies under its jurisdiction. Whichever document is used, the designer should be familiar with it. The discussion below only briefly mentions some inspection activities and priorities.

14.5.1 How Often?

On railroads and on parts of rail transit routes that are subject to FRA regulations, the frequency of inspection is typically based on the speed of trains, which in turn identifies a class of track. The FRA Track Safety Standards then identify an inspection frequency based on that track classification. No similar regulations apply to tracks that are not under FRA oversight. However, APTA's document RT-S-FS-002-02, *Standard for Rail Transit Track Inspection and Maintenance,*

stipulates that tracks used in revenue service should be inspected at least once a week and that other tracks should be inspected monthly. An interval of at least 3 but not more than 11 calendar days must elapse between inspections. In general, for the maximum speeds at which light rail trains typically operate, the FRA rules and the APTA standards are consistent concerning inspection frequency. However, rail transit systems that operate discrete track segments at greater than 60 mph [97 km/hr] might consider following the FRA requirement for Class 4 track, which requires twice-weekly inspections.

14.5.2 Priority Lists

This list is a valuable tool for the maintainer. One would like to think they could do everything right now, but that is not reality. Therefore, a priority list would be established and conveyed to the maintainers for action.

14.5.3 Qualifications

Both the FRA and APTA have developed guidelines to qualify people to inspect track and to oversee the performance of track maintenance tasks. The FRA has established a third classification for persons who are qualified to maintain and supervise work on CWR. This third category has a training component and experience levels since improper maintenance and inspection of CWR has led to serious accidents and fatalities on railways. Understanding how CWR reacts and how the forces due to temperature change and train dynamics must be restrained is imperative. The engineer must have a full and complete understanding of this also.

14.6 MAINTENANCE ACTIVITIES

The following discusses maintenance activities unique to each trackform. Writing a maintenance manual may sometimes be the responsibility of the designer. Since each agency may have its own unique character, some generic maintenance activities for certain trackforms will be discussed as listed below.

14.6.1 On-Track Maintenance-of-Way Equipment

A maintenance-of-way program requires appropriate equipment to fulfill its function. With the exception of LRT/streetcar lines that are 100% embedded track, much of this equipment will need to be designed to ride on the track. This will include hy-rail vehicles that can run on both the highway and the rails and machines that are rail-only. Because the equipment requirements for LRT maintenance can vary widely, this Handbook will not attempt to address the details of the many types of equipment that might be included in a comprehensive maintenance-of-way program. However, the following critical issues should be kept in mind when specifying and procuring M/W equipment.

- The maintenance vehicles must be compatible with the track. This seems obvious, but is often overlooked. The wheel contour and wheel gauge for M/W equipment should ordinarily be identical to that adopted for the light rail vehicles. More than one project has belatedly discovered that on-track maintenance equipment that was manufactured in strict accordance with AREMA recommendations doesn't fit a track structure designed

around transit parameters. Differences in back-to-back wheel gauge and the resulting incompatibility with narrow flangeways are distressingly common occurrences. Maintenance-of-way equipment with a long wheelbase may not be compatible with sharp radius curves. Since widening the flangeways to accommodate the rare operation of some machine with a long wheelbase would likely be detrimental to the compatibility of the track with light rail vehicles, it may be necessary to either prohibit such equipment from certain curves or procure an entirely different machine that is compatible with the track.

- The track must be compatible with the maintenance vehicles. This is typically an issue with hy-rail maintenance trucks. On one project, it was belatedly discovered that the light-duty material used for walking surfaces on pedestrian crossings was being crushed by the heavy rear tires of the maintenance department's hy-rail trucks. On another project, an extension to a system's existing light rail line, it was discovered that the hydraulic jacks on the transit authority's tamper would not clear the station platforms along the new track. Restraining rails that are elevated more than about ½ inch [13 mm] above the running rails can be problematic, as they can lift the rear tires of hy-rail trucks, which then lifts the rail wheels. Since this would occur in a sharp curve (hence the presence of the restraining rail), the rail wheels can then climb over the rail and derail.

14.6.2 Maintenance Issues Common to All Trackforms

14.6.2.1 Insulated Joints

Insulated joints are very important because they and the associated signal system circuitry prevent trains from hitting each other. Insulated joints define segments of track that cannot be occupied by more than one train. If an insulated joint fails, the signal system fails, albeit to a "safe" condition that simulates a train occupying the track.

Insulated joints must remain insulated so there is no continuity from rail to rail. Therefore, they should be checked periodically. Epoxy-bonded insulated joints are typical on LRT systems, but there may be standard bolted insulated joints also. Whichever type is encountered, they must be checked for loose bolts and broken or damaged insulation. The epoxy must be intact. There should be no gap between the end post and either rail.

14.6.2.2 Rail Corrugation and Rail Grinding

Rail corrugation and rail grinding are closely related topics that are necessarily addressed together.

14.6.2.2.1 Rail Corrugations

Rail corrugations are a form of rail wear that appears on the surface of the rail in a regular pattern. Corrugation is often caused by wheel slippage or dynamic creep. Wheels are tapered for proper tracking of the vehicle. Poorly matched wheel diameters on the same wheel set may cause slippage. This slippage usually happens in the same place all the time, particularly if the vehicle fleet is identical, which is often the case in rail transit. This slippage will cause very shallow "ruts" in the rail over time. These can cause noise and vibration, a phenomenon commonly called "roaring rail."

Corrugation is characterized by two main parameters. One is the length of the "waves." This is measured from crest to crest or valley to valley. Depending on the causes of the corrugation, the wave length can be anywhere between ½ inch [1 cm] to 3 feet [1 meter]. The other parameter is the depth. A good straight edge and taper gauge can measure this dimension. Each property should have some criteria for these parameters. In many cases, the people in the neighborhood will bring this roaring rail noise to the transit agency's attention.

The stiffness of the track (track modulus) plays a role in corrugation also. If the track is either too stiff or too soft, corrugations may develop. Finding the right stiffness for the loads imposed on the track is a challenge. The track designer should have a good understanding of this before selecting or specifying a rail fastener or setting the fastener spacing; see Chapter 9 for additional guidance on this topic. The track stiffness will be different in switch areas and the frog area also.

It is the maintainer's job to identify corrugation and implement corrective action before it becomes significant and causes wheels to jump or flutter. Left uncorrected, corrugations only get worse as their vertical amplitude increases. The resulting oscillations in the vehicle trucks will also cause the corrugation pattern to progress further and further down the track.

14.6.2.2.2 Rail Grinding—the Basics

Rail grinding is both an art and a science. Books have been written about it; however, very nearly all research has focused on rail grinding for open track on freight railroads. While much of that is applicable to any steel-wheel-on-steel-rail system, the smaller wheel loadings of rail transit change the parameters and hence the results that can be expected.

Rail grinding is commonly done to restore the original contour or make a new profile to better suit the wheels. The key to success is to match the wheel to the rail whereby the contact patch is optimized for proper load transfer. Besides the elimination of corrugations and the associated noise and vibration, other benefits to rail grinding include removal of rail defects that are initiated by rolling contact fatigue of the rail steel, lower rolling resistance, improved traction, and reduced energy consumption. While it seems counterintuitive, removing rail steel through grinding actually extends the life of the rail, because rail wear rates are reduced by optimizing the rail/wheel interface. Grinding will also produce better contact between the rail and wheel to both reduce electrical resistance for the negative return path and improve shunting for the signal system.

14.6.2.2.3 Rail Grinding Frequency

Rail grinding can be used for both preventative and corrective maintenance. Some studies suggest that a regular program of periodic rail grinding can extend rail service life by 30% or more.

Unless corrugations are ground out early, the work-hardened metal at the valleys of the corrugations will extend to significant depth below the running surface. Once that occurs, even if corrective rail grinding is performed, the corrugation pattern may be quickly reestablished. Eventually, the condition may become so intractable that the rail will need to be replaced at only a fraction of what its service life would have been had rail grinding been done early and often. In general, rail grinding done before the corrugations are plainly visible or audible is best, since it will prevent the steel from becoming work hardened beneath the corrugation

valleys. Careful monitoring of corrugation growth rates are necessary to determine an optimum grinding interval, which may be the time required for the corrugation amplitude to increase by a factor of two or three relative to some reference condition. Corrugation growth is largely exponential.

14.6.2.2.4 Specifying a Rail Grinding Program

Rail grinding, if done improperly, can cause more problems than it solves. This section simply offers some guidelines to keep in mind.

Very few transit agencies have the resources to own and maintain their own rail grinding equipment. Instead, most agencies procure rail grinding services from specialty contractors. This requires the preparation of a detailed contract specification. Whether the specification defines specific methods or is performance based, it must convey a message tailored to the needs of the individual LRT system. Since LRT systems vary a great deal, it is nearly impossible to suggest that one size fits all.

The owner's specifications should define the following:

- The characteristics of the rail system, particularly any factors that might impact the grinding contractor's equipment.
- The conditions under which the grinding program must be performed.
- The desired end product. This part could vary considerably depending on the owner's objectives, previous experiences with rail grinding, and the degree to which the owner wants to depend on the contractor's expertise to "do the job right."

Rail system characteristic factors that should be defined include the following:

- Trackform(s) and rail section(s) to be ground.
- Wheel profile and wheel gauge.
- Rail cant and track gauge.
- Maximum grades and minimum curvature. This is best addressed by including complete plan and profile drawings as an appendix to the specification.
- Details of track appliances found in the tracks to be ground. In particular, the contractor will be concerned about guard rail systems since they can affect the ability to position grinding stones around the gauge corner of the rail. Both restraining rail flangeways and the clearances to any emergency guard rails can affect the contractor's methods.
- Wayside clearances, particularly in tunnels and station platforms. Even low-level platforms can possibly obstruct equipment that was originally designed to work on a freight railroad.

Working condition requirements could include the following:

- Time of day limitations. Can track time be made available during the day or will all of the grinding need to be done on nights and weekends? How soon after the last train can the contractor get started? How much time must be allowed at the end of a work shift to clear the track?

- Contractor storage location(s). Like all contractors, the rail grinding contractor will need laydown space for the rail grinding train and off-track equipment and materials. The maximum traveling speed of most rail grinding trains is relatively slow, and the contractor will generally prefer to maximize his "spark time" by not running to and from the system's main storage yard at the end of his shift. Pocket tracks or spurs where the grinding train can be stored when not in use can significantly increase the contractor's productivity and decrease costs.

- Safety requirements, such as flagging of rail and automotive traffic.

- Environmental conditions. What are the maximum tolerable noise levels that will be allowed during the work? Will they vary by time of day and/or location?

- The rail surface finish tolerances they will be expected to produce. These metrics are often understated, especially by owners and consultants who have insufficient experience in rail grinding work.

- Will the third rail or overhead catenary be energized while the contractor is performing the work?

- What form of track protection will be in place?

- Must the grinding equipment have the ability to shunt the rails?

The rail grinding contractor should make several detailed submittals prior to mobilizing equipment to perform the work. Items that these submittals address could include the following:

- A detailed inspection report of the existing track, defining the existing condition of the rails with respect to corrugation and wear and highlighting any areas that may need special attention. This report may have been prepared in advance by the owner or a consultant who has been retained by the owner. Even so, the rail grinding contractor should be required to conduct an inspection and confirm whether this inspection corroborates the owner's or owner's consultant's findings.

- A detailed work plan describing how the contractor intends to perform the work. This would include the following:

    - Specifications on the equipment to be used, including clearance diagrams of the equipment to be used.

- Any limitations the equipment may have, such as the minimum radius the equipment can traverse and whether it can operate on non-standard gauge track.

- Environmental controls such as the dust collection, fire protection, and spark-arresting systems, as well as the system for collection and disposal of rail grinding debris. The debris might be considered hazardous material in some jurisdictions, and proper disposal is mandatory. The contractor should explain how the drainage system will be protected from becoming clogged with grinding stone particles and metal dust and also specifically explain how the metal dust will be prevented from breaching the signal system or creating a path for stray current.

- How the contractor intends to perform the work. This would include the size, type, and number of grinding stones to be used, including the grit size, the number of passes that will be made on each discrete segment of track, the pass speed, and the number of facets that will remain on the rail head at the completion of the work. The size of the grit in the grinding stones is an important issue for rail transit. Rail grinding on freight railroads can generally use stones with a fairly coarse grit since the heavy axle loadings will smooth the surface fairly quickly. The same coarse finish on a transit line might take months to wear away, especially with hardened rail. Moreover, during that wearing-in period, noise levels could actually be increased, and the periodic grinding marks in the running surface could initiate new patterns of corrugation.

- A schedule for when the work will be performed in each discrete segment of the route, including how the work will interface with rail transit operations.

- A description of and references for previous similar work performed on other rail transit properties.

14.6.2.2.5 Special Challenges for Transit Rail Grinding

There are many challenges that make grinding rail on a rail transit system distinctly different than similar work on a freight or passenger railroad. These include the following:

- Rail grinders will produce sparks that can cause fires or damage, including eye injuries if bystanders are watching the work. Since LRT rights-of-way are typically much closer to the general public than freight railroad tracks, special measures are necessary to protect the public.

- Rail grinding is a noisy operation. This issue must be addressed as part of the bid documents for procurement of rail grinding services; otherwise, the operation might be shut down on the first night by neighbors' complaints.

- Rail grinding can be especially challenging on metro rail systems that use a third contact rail for traction power distribution since metallic grinding dust can cause leakage around insulators on both the negative and the positive sides of the traction power circuit.

- Rail grinding in embedded track has unique challenges; see Article 14.6.3.8 for a detailed discussion of these issues.

14.6.2.3 Repair Welding and Grinding of Switches and Frogs

This is another important activity that will allow the switch points and frogs to achieve the maximum life possible. Monitoring wear of frogs and switch points should be performed by the maintainer. Periodic measurements will facilitate predicting when the frog or switch point will be worn more than the criteria given in the maintenance standards manual. The measurements must be taken at regular intervals such as every 3 to 6 months. Switch points and frogs can be welded back to their original shape using the correct material and workmanship. This is a job for professionals who have been trained to weld switch points. If an untrained individual performs this work, there is a high probability of weld defects that can propagate and develop into cracks that can cause the tip of the switch point to break off completely and create an obvious high risk for derailment.

14.6.2.4 Emergency Repairs

It is important that these activities be identified in the system maintenance standards. What constitutes an emergency? Wear on rail is not an emergency. A track buckle is an emergency! Every reasonably possible problem or occurrence should be identified in the maintenance standards and categorized as to whether it should be considered an emergency or not. Risk analysis can play a very important role in deciding the definition of an emergency.

Each emergency must have an emergency response plan designating what actions are required and/or are allowable, or are prohibited. These are decisions that the property owner must resolve based on local conditions. For a new rail system, where the owner may not yet have experienced rail personnel on staff, the track designer may be the owner's best resource for making these decisions.

A few examples of situations that might be addressed in the plan include the following:

- In an emergency situation, it is generally accepted that a torch may be used to cut the rail, but specific conditions might rule against that.
- Designating the number of fasteners in a row must become ineffective before an emergency is declared.
- Designating how far back a switch point must break before an emergency is declared.

It is imperative that these decisions are made prior to commencement of revenue service.

14.6.2.5 Maintenance Documentation

Records of inspection and maintenance activities should be kept up-to-date and contemporaneous with the work performed. Such records can make future maintenance easier, but the more important reason to keep comprehensive records is as a legal protection. Records are needed to be able to prove that inspections and maintenance have been performed on a timely basis. The following agencies and organizations have certain levels of regulation and/or oversight with respect to rail transit, including maintenance documentation. It is strongly recommended that each designer be familiar with salient documents published by these concerns.

14.6.2.5.1 The Federal Railroad Administration

The Federal Railroad Administration (FRA) regulates trackwork that is part of the general railroad system of transportation and has some oversight of tracks that, while they are not part of the general railroad system, are proximate to FRA-regulated tracks. While much of the FRA's specific inspection and maintenance requirements may not apply to a rail transit line, FRA's requirements for record-keeping, as published in 49 CFR 213.241, represent good practice that should be considered during the development of a system maintenance plan.

14.6.2.5.2 The American Public Transportation Association

The American Public Transportation Association (APTA) has written safety standards which can be downloaded from its website. It consists of six volumes entitled:

(1) Background and Process

(2) Vehicle Inspection and Maintenance

(3) Rail Grade Crossings

(4) Operating Practices

(5) Fixed Structure Inspection and Maintenance

(6) Signals and Communication Inspection and Maintenance

The APTA documents are generally in the same format as the FRA regulations but also include a color-coded priority system to offer guidelines on severity. The APTA documents are voluntary standards and, as noted earlier, are necessarily generic because of the wide variation in transit system infrastructure and systems from one transit agency to another.

14.6.2.5.3 The Federal Transit Administration

The Federal Transit Administration (FTA) has the task of overseeing transit agencies, but, unlike the FRA, the FTA does not (as of 2010) have the authority to govern transit operations or to levy fines. The FTA principally administers funds to build and maintain transit systems. In the process of administering those funds, the FTA does have the discretion of requiring compliance with standards, codes, and requirements developed by other public agencies and private entities.

14.6.3 Embedded Track Maintenance Requirements

14.6.3.1 Cleaning the Flangeways

Unless the track gradient is steep enough so that ordinary storm water runoff will flush debris out of the flangeway, it will fill up with all sorts of street detritus. While there is a slight danger of this material causing damage to the LRV wheels, the real danger is that the rail insulation system (typically a rail boot) will be compromised and that trace amounts of stray current will be able to leak off the rail over an extended length of track. It might also affect train control system circuitry if track circuits are used. This is a situation where groove rail has an advantage over tee rail with a formed flangeway. Frozen debris in a flangeway could also cause a derailment.

Avoiding very flat grades and providing frequent track flangeway drains are measures that the track designer can take to make certain the flangeways remain clear. When flat grades are unavoidable, the owner should be advised that additional maintenance effort should be expected (and budgeted for) to keep the flangeways clear.

At one LRT system in Germany, a specially equipped rail grinding vehicle also includes a flangeway scraping tool with a vacuum system for cleaning the flangeways of groove rails. Such a system might not work as well on embedded tracks using tee rail because the scraping tool might damage the pavement and the rail boot or other rail isolation system. The LRT system in Calgary has a vacuum truck that is designed specifically to clean flangeways.

14.6.3.2 Repair Top Surface

Spalls and cracks in the roadway surface should be repaired/sealed and not left open for long periods of time. This is another avenue for water penetration into the underlayment, which can cause destruction of the embedded track system over time.

14.6.3.3 Spotting Stray Current

Learning the signs of stray current leakage and corrective actions to be taken early can save a system the cost of a major track reconstruction. It may begin with spalling of the concrete surface or false indications to the signal system. Observing minor arcing during wet weather at night could be an indicator. Track drains are likely locations for stray current leakage and should be inspected and cleaned frequently. While stray current may take years before it becomes a hazard, regular inspection and preventative maintenance are recommended. A professional corrosion consultant should be retained to test the system annually and identify any “hot spots” so that they can be corrected before they cause severe damage.

14.6.3.4 Snow Removal

When a rail vehicle encounters snow or ice on a rail head, the extremely high contact pressure between the wheel and rail will typically cause the ice to melt despite the cold temperature. This is the same phenomenon that enables a figure skater to glide across the ice. Because of this, light rail vehicles can be as sure-footed through light snowfalls as they are in the rain, particularly when equipped with sanders. However more than 2 to 4 inches [about 5 to 10 cm] of snow over the rails can be problematic, particularly on steep grades or if ice has filled the flangeways. Accordingly, removal of snow and ice from embedded tracks is an important activity. Typically, ordinary snowplow trucks can be used although some systems have used locomotives equipped with plows. Legacy streetcar systems often had snow sweepers, but no such vehicles are currently in use in North America. Snow that is contaminated with dirt, cinders, and de-icing chemicals can also compromise track insulation systems, lead to erratic signal system operation, and become a path for stray current.

14.6.3.5 Repair Broken Rail

While a broken rail in embedded track is generally not as severe a hazard for derailments as one in open track, it interrupts the traction power negative return and hence could result in a major stray current problem at an entirely separate location. Broken rails therefore should be promptly repaired. Even if a full repair isn’t immediately possible, a rail bond cable should be installed around the break until a full repair can be made.

Repairing a broken rail (or a worn-out rail) in embedded track is tedious and expensive as it requires concrete removal over a significant distance so a new rail plug can be installed. The integrity of the rubber rail boot or other isolation system must be restored before the pavement can be restored.

The concrete surface must be saw cut on either side of the rail a distance that will expose the fastening system when the concrete is removed. The concrete is then chopped out to a depth near the base of rail. The elevation is already preset by the elevation of the existing concrete under the rail; therefore, the concrete can be rough adjacent to the base. The fasteners are exposed and can be removed. The boot can be cut back to where the rail is to be cut, allowing enough room for a repair cuff to be installed. The thermite weld and the rail boot repair cuff will both require concrete demolition to well below the base of rail. Hydro-demolition is sometimes convenient for that purpose, but the area must be thoroughly dried before the thermite weld is made. After the rail is cut, the old rail can be removed as well as the old boot. The concrete surface should be cleaned at this time and a visual inspection performed. Any damaged coatings on exposed reinforcing steel will need to be patched. The new piece of rail can be booted and set into the seat area established by the old rail. Fasteners can be reinstalled, welds made, and the boot cuffed and tested. After a quality control check, the concrete can be replaced and revenue service restored.

14.6.3.6 Replace Worn-Out Rail

It generally takes a very long time for rail to wear out in the LRT environment. Significant wear will usually be observed only in sharp curves or at locations where heavy braking occurs. Stations/stops are usually locations of accelerated top-of-head rail wear due to occasional use of track brakes. The rail replacement issue led to some early embedded track designs that tried to facilitate rail change-out by making it easy to access the base of the rail. The problem with such designs is that the features that make it easy to access the rail fastenings also make it easy for water and contaminants to access them and possibly initiate corrosion. The use of a harder grade of steel rail is usually a better investment since it makes it far less likely that the rail will wear out prematurely. Ideally, the service life of the rail will match or exceed the service life of the embedding pavement.

If a section of embedded rail needs to be replaced because of wear, the process is generally as described above for a broken rail.

14.6.3.7 Replacement of Parts in Embedded Specialwork

Advance planning for replacement of parts in embedded special trackwork must be incorporated in the design and fabrication. The most likely parts to require replacement are the switch tongues and the switch machines, including the connecting hardware to the tongues. The procurement specifications need to include requirements that it should be possible to replace those items without demolition of pavement. Ideally, this work should be possible using ordinary hand tools to remove and reinstall fastenings and a light capacity crane to remove and reinstall the major components.

14.6.3.8 Rail Grinding in Embedded Track

Because flangeways in embedded track (as well as in grade crossings along open trackforms) are so narrow, a rail grinding contractor must use different rail grinding stones in embedded track that may require an entirely different methodology than with open trackforms such as direct fixation and ballasted. It is difficult to grind rail in embedded track and conventional grinding equipment may not work properly. The development of longitudinal belt sanders may be the answer for embedded type track.

Many light rail systems in Europe have specially equipped rail vehicles for performing preventative maintenance rail grinding. These cars are often older high-floor articulated vehicles that are no longer being used for passenger service. The center truck on such cars is equipped with grinding blocks that take the place of the magnetic track brakes. Hydraulic pistons can maintain a constant downward pressure on the grinding stones. These rail grinding cars are run over the system on a regular schedule, often running at relatively high speeds between revenue service trains during daylight hours. The cars are often equipped with tanks to spray cooling water on the grinding stone as well as a vacuum system for collecting the spray water and any grinding residue. The routine use of such block grinding cars has been proven to defer the need for corrective grinding with conventional rotary rail grinding equipment and, in some cases, prevents corrugations from ever reaching a level where corrective grinding is even necessary.

Curiously, many legacy North American streetcar systems used similar equipment in times past, but the practice fell out of favor for reasons that are no longer clear. It may have been simply a matter of the equipment getting old and difficult to maintain. As of 2010, at least one North American LRT system was going to be procuring modern block grinding equipment based on current European practices.

14.6.4 Direct Fixation Track Maintenance Activities

Direct fixation track is an “open” trackform with nearly all of the major components easily visible and accessible for inspection and maintenance. Other open trackforms include ballasted track, some forms of grass track, and open deck bridge track. The paragraphs below will discuss various maintenance activities that are applicable to direct fixation track and may also be applicable to other trackforms as well.

14.6.4.1 Tighten Bolts

While the use of CWR eliminates most bolts in rail, direct fixation track is loaded with other bolts. These include rail fastener anchor bolts, emergency guard rail bolts, and numerous nuts and bolts within special work. Systems incorporating a contact rail instead of overhead catenary will have dozens of other types of bolts. Every one of these bolts is subject to loosening. If one bolt becomes loose, the adjacent bolts must restrain the dynamic forces triggering them to loosen or break in turn. New track is particularly prone to bolt loosening until such time as mating parts wear in and seat to each other.

The designer can assist in this by avoiding designs that incorporate threaded fastenings in inaccessible locations where both inspection and tightening are difficult.

14.6.4.2 Rail Clip Maintenance

Elastic rail clips, which are fabricated from spring steel, require regular inspection and occasional replacement. Since parts of the clip are highly stressed, they can be particularly susceptible to corrosion and sudden failure. Exfoliated rust from the rail clips has been known to become impacted between the clip and the clip housing, making it impossible to remove the clip (or the broken stub of a clip) from the housing. This sort of problem is common within damp tunnels.

Designers should carefully consider whether particular trackway environments might not be suitable for some products (such as a particular rail clip design) due to environmental conditions, loadings, or both. Prior to soliciting bids for such items, the service environment where the product will be used should be made very clear to potential vendors and their advice solicited concerning the most appropriate products to be used. The manufacturer's recommendations concerning installation, inspection, and maintenance should be incorporated in the system maintenance manuals and closely followed by maintenance staff.

14.6.4.3 Housekeeping

Direct fixation track can provide extremely long service if properly constructed and maintained. A significant part of that maintenance is simple housekeeping. Trackways are dirty environments and that dirt needs to be washed off the track at frequent intervals. For ordinary direct fixation track out in the open, normal precipitation is sufficient to keep the track clean. But tracks in tunnels obviously do not see rainfall and tend to accumulate all sorts of dust and dirt. If the tunnel is damp, this material tends to be corrosive and attacks the rails, rail fasteners, and other appurtenances.

LRT tunnels in regions that get snowfall and see heavy use of de-icing chemicals are particularly susceptible to extremely severe trackway corrosion. The LRVs will pick up corrosive slush at road crossings and embedded track segments and carry that material into the tunnels where it will melt and land on the track system. This corrosive brine not only initiates corrosion chemically, it provides paths for stray currents, causing even more accelerated corrosion. Several LRT systems have needed to completely replace rails and rail fasteners in such tunnels, while identical construction in other areas of those systems that were not subject to brine attack remain in nearly new condition.

The primary defense against this sort of problem can be relatively simple. Completely washing tunnel trackwork with the equivalent of a "fire hose" on a regular basis (say twice a year) can remove the brine and other contaminants before they can cause significant corrosion. The debris can be washed toward tunnel drains where solid particles should be collected rather than being passed through to municipal sewer systems.

Direct fixation tracks in LRT stations tend to accumulate traction sand beneath the rails and between the rail fasteners, sometimes mixed with lubricants that drip from the vehicles. This compound can also cause corrosion and stray current problems and should be power washed away on a regular basis.

14.6.4.4 Regauging Track for Rail Wear

When the rail begins to wear on curves, it may be necessary to adjust the gauge in order to stay within maintenance standards for gauge. If "zero adjustment" fasteners were installed during construction, then it is not possible to do this without core drilling and moving the inserts; however, most fasteners have adjustments built into the anchorage assembly for adjusting gauge and line up to ½ inch each way from center. Provided the constructor didn't use all of this adjustment during construction, the maintainer should be able to correct the gauge. Preserving this capacity makes it very important that the contractor center the anchor assemblies during construction. As noted in Chapter 13, the use of top-down construction greatly facilitates this by preserving virtually all of the lateral adjustment capacity.

14.6.4.5 Repair Cracked and Spalling Concrete

Cracks happen, and, as noted for embedded track, they should be sealed so no water enters the interior of the concrete. If this occurs, then water may begin to rust the reinforcing steel; more importantly, this gives a path of least resistance for the stray current. In colder climates during freeze/thaw cycles, the frozen water could spall the concrete, leaving a large exposed area. Fixing minor cracks is relatively easy—a small handheld chipping gun is used to chisel or "V" out an area along the crack. Then caulking or some other flexible watertight material can be used to seal the crack.

14.6.4.6 Repair Caulking

Some designs require that a bead of caulking be placed around the plinth. This is primarily to prevent water from getting under the plinth and causing the same problems that may be encountered from spalls or cracks. If the original caulking becomes ineffective, it should be removed. The area must then be prepared according to the caulking manufacturer and a new bead of caulk applied.

14.6.4.7 Repair of Broken Rail

When a rail breaks, it is usually because of excessive tensile force that has built up in the rail due to cold weather or train dynamics. If this occurs in direct fixation track on an aerial structure, significant problems could ensue due to unbalanced forces among the structure, the broken rail, and other unbroken rails on the bridge. It must be understood that when such a system is built, the interface between the guideway and the rail is correctly matched and should not be changed. Therefore, it is imperative that no rail be added.

The main issue when repairing any CWR break is to not change the neutral temperature of the rail. Simply welding in a new rail plug will cause problems when the rail gets hot. In ballasted track and also in direct fixation track on a structure, there could be major problems if the neutral temperature of one rail is significantly different from the other rail on the same track.

The CWR plan that every track owner must have needs to explain the procedure for correcting this defect. Typical plans for the correction of a broken rail are several pages long, but the bottom line is to pull the rail back together after removing at least 200 feet [60 meters] of rail clips each way, then weld the rail back together. In general, there should be criteria for breaks less than 3 inches [75 mm] and more than 3 inches. If the break is more than 3 inches, a plug rail must be put in, but the CWR's zero stress temperature must be readjusted prior to the return of warmer spring weather.

It is also imperative that these activities be documented. If a rail break occurs near this location again, then the rail may need to be de-stressed again for a much longer length. Refer to Chapter 13 for more detail on adjusting CWR for optimal zero thermal stress.

14.6.4.8 Replacing Rail Fasteners

Occasionally, a fastener may break or become ineffective. Maintenance standards should address how many rail fasteners in a row or within a section of track may be defective just as standards do concerning defective crossties.

Replacing an individual fastener that is anchored to female inserts embedded in the plinth is a relatively easy task. Simply remove the clips and hold-down bolts. Raise the rail after removing a few clips on either side. Then, slide the fastener out and install a new one. Lower the rail, reclip

every fastener, and torque the hold-down bolts to the correct torque. While the fastener is not there, it is wise to inspect the concrete under the fastener to determine what caused the fastener to fail. If any voids are found under the fastener, they should be repaired before installing the new fastener, or the new one will become broken in a period of time.

Sometimes it may be determined that the anchor inserts sit above the plane of the concrete surface, causing point loads directly on top of the inserts. If this is the case, the top of the inserts should be ground flush with the surface of the concrete and then epoxy paint applied to the bare steel.

Fasteners that are anchored using threaded rods rather than female inserts require that the rail be raised very high in order for the defective fastener to be removed. This requires the unclipping of much more rail, which could result in the rail bucking in hot weather when compressive forces in the rail are high. For this reason alone, the use of female inserts rather than threaded rod anchors is preferable. In all cases, it is wise to do this activity when the rail temperature is below the neutral temperature. In the maintenance manual for the system, stipulate the maximum number of rail clips in a row that may be removed.

Several rail transit systems have needed to perform out-of-face replacement of long segments of direct fixation rail fasteners, usually in track segments constructed in the 1970s or 1980s. Many of the rail fasteners used at that time were inferior designs that did not hold up well under service. These out-of-face replacement projects have more in common with new construction than with ordinary maintenance work.

14.6.4.9 Rail Fastener Anchor Insert Replacement

Occasionally, for whatever reason, an insert may fail or the threads become ineffective. If this happens, then the insert must be removed and a new one put in. The replacement process is to remove the defective insert by core drilling and grouting a new insert in its place. Once the rail fastener is temporarily removed, core drills can make this work relatively quick and easy. It is very important to ensure that the new insert is perpendicular to the bottom plate of the fastener, is either even with or slightly below the surface of the concrete, and is not in contact with the sides of the cored hole. Only then should the insert be anchored in place, typically with a two-component epoxy grout. Either templates or the fastener itself should be used as a guide for positioning the new insert. It is very important to follow the grout manufacturer's recommendations, particularly environmental controls such as temperature and keeping the work area dry.

14.6.4.10 Replace Continuous Welded Rail

It is important to ensure that no rail is added or subtracted which will change the neutral temperature and cause unnecessary forces to be imposed onto the guideway. Track time is always a concern for doing this activity. The fasteners will play a role in the speed and effectiveness of performing this activity. If female inserts are used with bolts to attach the fasteners to the invert, only the rail clips will need to be removed and the new rail installed. If threaded rods are used to anchor the rail fastener body to the invert, they frequently project some appreciable distance above the fastener. If so, the rail must be lifted over the rods when removing it from, or placing it in, the fastener seat since impact could render the rods unusable

Many rail transit maintenance organizations, particularly on smaller systems, will not have the proper equipment and trained personnel necessary to correctly and efficiently perform out-of-face replacement of CWR. In such cases, it may be an activity best assigned to a qualified track contractor.

14.6.4.11 Replace Parts in Special Trackwork

There are numerous pieces and parts that make up a specialwork portion. The owner should have a full complement of replacement parts, and, when one is used, an order for restocking should be immediate. As the inspector walks each special portion weekly, they will generate a list of deficiencies. These lists should be monitored periodically to confirm compliance with maintenance standards. The challenge for the supervisor is to determine when corrective action should take place. In any case, corrective action should take place before the defects become too many. Minor bolt tightening and clip adjustment should be done on a weekly basis. Some parts such as frogs, switch points, and guard rails may require a contractor with the equipment to perform the work.

14.6.5 Ballasted Track Maintenance Activities

Ballasted track, like direct fixation track, is an "open" trackform; thus, many of the maintenance activities described above for direct fixation track will also apply to ballasted track. The paragraphs below will address some of the activities that are unique to ballasted track.

14.6.5.1 Ballast Maintenance

Ballast is an important part of the track structure for controlling thermal forces. The correct cross section of ballast therefore must be maintained in order to keep the track stable. Ballast may begin to migrate away from the ties, and therefore must be replaced or moved around. If tamping is to be done, it is advisable to place ballast first in excess to accommodate the tamping cycles. That way the track does not become unstable during the tamping operations. The track inspector should always look for areas with weak ballast, and any such area should be repaired as soon as possible, particularly during hot weather.

Ballast cleaning is a common activity on freight railroads but is rarely undertaken on rail transit lines. The best approach to avoidance of any need for ballast cleaning or replacement is to use a good hard ballast stone to begin with and to then keep up with basic housekeeping, including maintenance of drainage systems.

14.6.5.2 Surface Track

Ballasted track can be expected to settle and shift over time. This is a normal thing and actually helps track stability so long as it is uniform and does not create any abrupt horizontal or vertical misalignments. When this happens, it is necessary to raise and tamp the ties to restore a good surface. Extra ballast is placed first in the area of the track that has settled. Then the track can be raised and tamped without affecting the neutral temperature or the stability of the track. The track looses stability just after it is raised, so operating rules must be followed with respect to a slow speed order after maintenance has been performed. Alternatively, a stabilizer could be used to restore the track modulus.

14.6.5.3 Repair Deflector Ramps at Crossings

Every crossing should have deflector ramps that prevent dragging debris from abruptly getting caught on a sharp edge and causing damage. These ramps should be repaired immediately if damage occurs.

14.6.5.4 Replace Pads and Insulators on Cross Ties

Over time the pads and insulators that are part of the rail fastening assemblies on concrete cross ties will wear, crack, or otherwise become defective. This could compromise both the structural integrity of the cross tie and the electrical isolation of one rail from the other. As explained above for replacement of direct fixation rail fasteners, care must be taken so as to avoid having the rail buckle. It is generally inadvisable to do this work when the rail is above its neutral temperature or outside of the working range identified by maintenance procedures. Working in very short sections or isolated locations should not affect the neutral temperature or hurt the stability of the track. It is good practice to replace the insulators at the same time the pads and or rail clips are replaced.

14.6.5.5 Replace Cross Ties (Timber and Concrete)

Wood ties become rotten and damaged, and concrete ties become broken or abraded. When this occurs beyond the limits of the track standards, then the ties should be replaced. A good tie maintenance program is critical to the cost-effectiveness and safety of the railway. During the weekly inspections, a track walker may identify certain ties that are defective. The supervisor must plan tie replacement activities properly.

Replacing timber ties is relatively simple because once the spikes have been pulled, the tie can be pulled out to the side without raising the rail. This assumes there is sufficient lateral clearance on one or both sides of the track to extract the old tie and insert the new one. If not, it may be necessary to remove ballast and then respace several ties tightly while the defective tie is rotated 90 degrees and lifted out from between the two rails. The new tie is then installed, the tie spacing is restored in the reverse order, the ballast is reinstalled, and all disturbed ties are tamped. This is a tedious process that requires a lot of manual labor and, hence, is very expensive. For this reason, curbed ballasted track sections are undesirable. So as to minimize future tie replacement costs, it should be possible to extract and replace ties from at least one side of the track, preferably without fouling the other track.

Concrete tie replacement can be more challenging because the shoulder embedded in the concrete tie typically requires the rail to be raised by 2-inches [50 mm] so the tie can be extracted. The rail clips on the adjacent ties will need to be removed so the rail can be lifted without disturbing the bedding of the adjacent ties in the ballast, thereby risking “humping” the track. As with timber ties, spot replacement of individual ties in a curbed ballast section can be tedious and expensive, especially since the heavier concrete ties cannot be handled by manual methods.

14.6.5.6 Vegetation Control

Vegetation must be kept in check so it does not interfere with the operator’s line of sight, particularly the view of signals, crossings, and any areas where pedestrians might be on or near the track. At highway crossings, vegetation control needs to be more expansive so that sight triangles in the crossing quadrants are not obstructed. Vegetation should not block any signs or

signals and must not interfere with track inspection or maintenance. There are professional services that can be contracted in order to keep vegetation from becoming a defect. The track alignment designer should attempt to configure crossings so that the sight triangles remain clear. Sometimes, other disciplines on the design team, not understanding the safety and maintenance issues involved, will wish to plant vegetation in locations where it would become a problem for the maintenance organization.

14.6.5.7 Drainage Maintenance

Drainage is a very important aspect of good track maintenance. The water must flow, and the drainage system must be capable of handling the unexpected. It is considered a track defect for any drain pipe to be clogged or even just partially blocked by an object such as a tie, a pile of tie plates, or simply trash. Since most rail transit systems are built in urban areas, trash can be a problem and must be cleaned up promptly. APTA suggests that rail operations should be suspended when water levels get up to the ball of the rail. Even water levels at the base of rail can play havoc with the signal system and also make for problematic conditions concerning traction power return currents. Ditches, culverts, cross drains, under drains and other drainage systems therefore must be kept free of debris. Railway/highway crossings can be very susceptible to flooding if the pavement slopes toward the track and the designers didn't include sufficient drainage infrastructure to intercept storm water before it gets to the track. In direct fixation track, blowing debris that can get caught in track and signal appliances can block drainage. Such blowing debris can also become a fire hazard, particularly on a metro rail system powered by a third rail. Understanding that trash will blow in the direction of train traffic is a simple concept, and "trash catchers" bolted to the invert in stations can intercept debris before it can reach a location where it can cause greater problems.

14.6.5.8 Yard Cleaning

Yards tend to be collectors of trash and debris and must be cleaned also. Most yards are where workers will clean the vehicles, and workers will congregate before going out on the system. Good housekeeping in yards is also important and will set the tone for workers' attitudes. There are certain forms of equipment that are designed to clean trash. They may blow the trash to a collection point or vacuum it into a collection container.

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Abbreviations and acronyms used without definitions in TRB publications:

AAAE	American Association of Airport Executives
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACI–NA	Airports Council International–North America
ACRP	Airport Cooperative Research Program
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
HMCRP	Hazardous Materials Cooperative Research Program
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
NASA	National Aeronautics and Space Administration
NASAO	National Association of State Aviation Officials
NCFRP	National Cooperative Freight Research Program
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
PHMSA	Pipeline and Hazardous Materials Safety Administration
RITA	Research and Innovative Technology Administration
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation

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FINAL REPORT

TCRP Project D-14 [Report 155]

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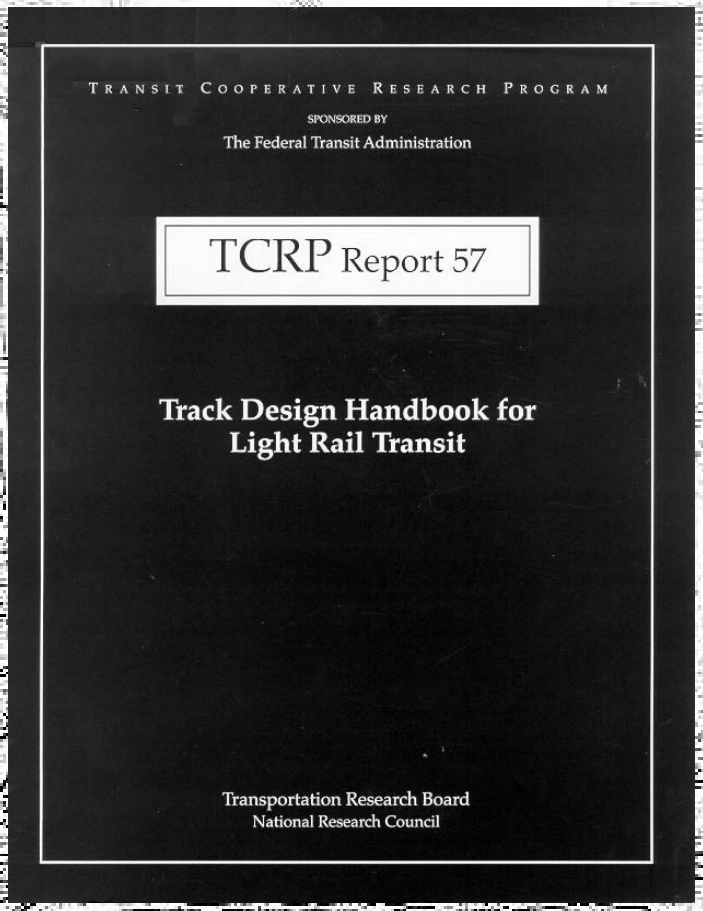
Track Design Handbook for Light Rail Transit 2nd Edition

PARSONS BRINCKERHOFF

Prepared by:
Lawrence G. Lovejoy, P.E.

TCRP Report 57



Updating Required

- Over a decade since original publication
- Doesn't reflect other TCRP research since then
- Streetcar issues not covered
- Construction & Maintenance not covered in a comprehensive way.

The D-14 Project Research Team

Parsons Brinckerhoff

Wilson Ihrig & Associates

Metro Tech Consulting

Track Guy Consultants

4.2.2 Standard Track and Wheel Gauges

The majority of contemporary rail transit systems nominally utilize "standard" track gauge of 1435 mm (56-1/2 inches). This track gauge stems from 18th century horse drawn railways used by English collieries, where track gauge was dictated by the common wheel-to-wheel "gauge" of the wagons used to haul the coal. This wagon gauge can be traced back to ancient times, where it was used on Roman chariots because it approximately matched the center-to-center distance of a pair of war horses. This made it easier for the horses to follow the wagon ruts in the roads. While many different track gauges were adopted over the years, none have proven to be either as popular or practical as standard gauge.

Track that is nominally constructed to standard gauge can actually be tighter or wider than 1435 mm depending on a variety of circumstances. The track gauge can be adjusted along the route so as to optimize vehicle-to-track interaction. Conditions that can require gauge adjustments include track curvature, the presence or lack of curve guard rails, rail cant, and several vehicle design factors. Vehicle factors include wheel diameter; wheel tread taper and width; wheel flange shape including both height and thickness; the distance between axles; and the wheel gauge or distance between wheels mounted on a common axle.

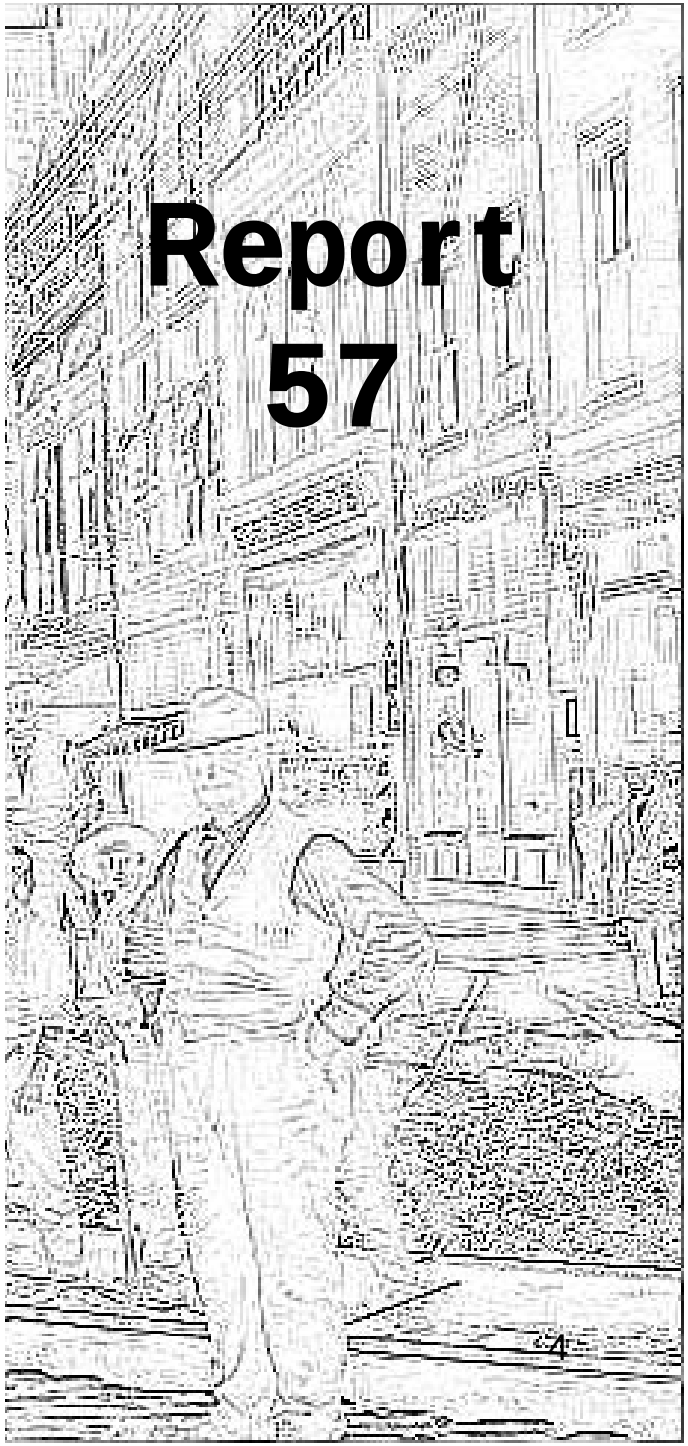
While nominal standard gauge is nearly universal for both electric rail transit and "steam" railroads, different requirements of these modes resulted in appreciably different details, such as where the track gauge is measured, under what conditions it is varied, and the amount of freeplay that is required between the wheel flanges and the sides of the rails.

4.2.2.1 Railroad Gauge Practice

North American railroads set track and wheel mounting gauges in accordance with criteria established by the Mechanical Division of the Association of American Railroads (AAR) and the American Railway Engineering and Maintenance-of-Way Association (AREMA). AAR standard wheel gauge is defined as 55-11/16 inches (equivalent to 1,414 millimeters) and is measured 5/8 of an inch (15.9 millimeters) below the wheel tread surface. The AREMA definition of track gauge is measured at the same distance below the top of rail. These gauge standards have been incorporated in many contemporary LRT track designs to accommodate possible joint railroad and LRT operations.

If wheels using the current AAR-1B wheel profile are mounted at standard AAR wheel gauge, and the wheel and axle assembly is centered between the rails at standard track gauge, the horizontal clearance between the wheel and the rail at the gauge line elevation is 13/32 inch or 10.3 millimeters as shown in **Figure 4.2.1**. This results in total freeplay between correctly mounted and unworn wheelsets and exactly gauged rails of 13/16 inch or almost 21 millimeters.

It is important to recognize that railroad gauge practices generally evolved in a different environment than transit operations. Particularly in curved tracks, railroad criteria is predicated on the use of equipment that generally has much larger diameter wheels than those used on transit vehicles. In addition, both the maximum wheelbase and the number of axles that might be mounted on a rigid truck frame are usually much greater. Steam locomotives in particular could have wheels over 1800 millimeters (6 feet) in diameter, with up to five such sets of wheels on a rigid frame. Even contemporary diesel



Changes From Report 57

Chapter 1 – GENERAL INTRODUCTION

- **Increased focus on who the Handbook user is and how he/she fits within an LRT project.**
- **Expansion of the definition of what LRT is.**

Chapter 2 – LIGHT RAIL TRANSIT VEHICLES

- **Updated discussions about characteristics of contemporary LRVs and Streetcars**
- **Added and in-depth discussion of how to select an appropriate wheel profile.**

Changes From Report 57

Chapter 3 – LRT TRACK GEOMETRY

- **New discussion provided explaining the relationships between lateral acceleration, unbalance and the jerk rate**
- **Expanded discussion concerning track crosslevel and limitations on twist in tangent track.**
- **Expanded discussion of alignment of freight tracks and shared LRT/Freight tracks.**
- **New article on issues where LRT shares a corridor (but not tracks) with a freight carrier**

Changes From Report 57

Chapter 4 – TRACK STRUCTURE DESIGN

- **The entire discussion about Nytram plots, Filkins-Wharton diagrams and related issues restructured to introduce basic concepts and background information first and only then develop the explanation of the methodology.**
- **Discussions about restraining rails and their application has been greatly restructured and expanded with reference to TCRP D-7 project.**

Changes From Report 57

Chapter 5 – TRACK COMPONENTS & MATERIALS

- **Tee rail discussion revised.**
- **Groove rail discussion completely rewritten.**
- **Restraining rail discussion completely rewritten.**
- **Rail Fastener / Rail Fastening discussion relocated and rewritten**
- **Crosstie discussion largely rewritten**

Changes From Report 57

Chapter 6 – SPECIAL TRACKWORK

- **Expanded discussion on types of frogs including flangebearing and jump frog designs.**
- **Revised/expanded discussion on selection of turnout size.**
- **Revised and expanded discussion on switches, including those for embedded track.**

Changes From Report 57

Chapter 7 – STRUCTURES & BRIDGES

- **Rewrote discussions of various types of LRT structures and how they work.**
- **Entirely new discussion on structures carrying embedded track.**
- **Entirely new discussion on open deck structures and retrofitting existing highway bridges to carry LRT track.**

Changes From Report 57

Chapter 8 – CORROSION CONTROL

- **Miscellaneous revisions emphasizing inter-disciplinary coordination issues.**

Chapter 9 NOISE & VIBRATION CONTROL

- **Greatly expanded with extensive text revisions addressing salient issues including wheel/rail contact mechanics. Extensive new graphics**

Changes From Report 57

Chapter 10 – TRANSIT SIGNAL WORK

- New text on embedded switch machines.
- Expanded text on rail bonding for signal and power current, including reference to TCRP D-7 work

Chapter 11 – TRANSIT TRACTION POWER

- Expanded discussion on how track alignment affects OCS design.

New Chapters for 2nd Edition

Chapter 12 – LRT TRACK IN MIXED TRAFFIC

- **Entirely new chapter with emphasis on track alignment for streetcar style operations.**

Chapter 13 - LRT TRACK CONSTRUCTION

- **Entirely new chapter with emphasis on how LRT tracks of all types are actually built.**

Chapter 14 – LRT TRACK MAINTENANCE

- **Entirely new chapter with emphasis on how LRT track maintenance is typically organized and how it can be directly affected by design decisions.**

Changes From Report 57

Reversed Metric vs. US Traditional Measurement Units

- **Report 57 was prepared under the old Metrification Policy.**
- **Second Edition reverses the priority of SI vs. US Traditional units of measurement**
- **US units now first, with SI units [in brackets]**

EXCEPTION:

- **When the dimension being discussed is metric in origin (principally for manufactured products) the SI units are listed first followed by a soft translation to US Traditional Units.**

Challenges that Remain

#1:

How can we keep the *Track Design Handbook for Light Rail Transit* up-to-date?

For questions concerning this presentation or for further information / assistance contact:

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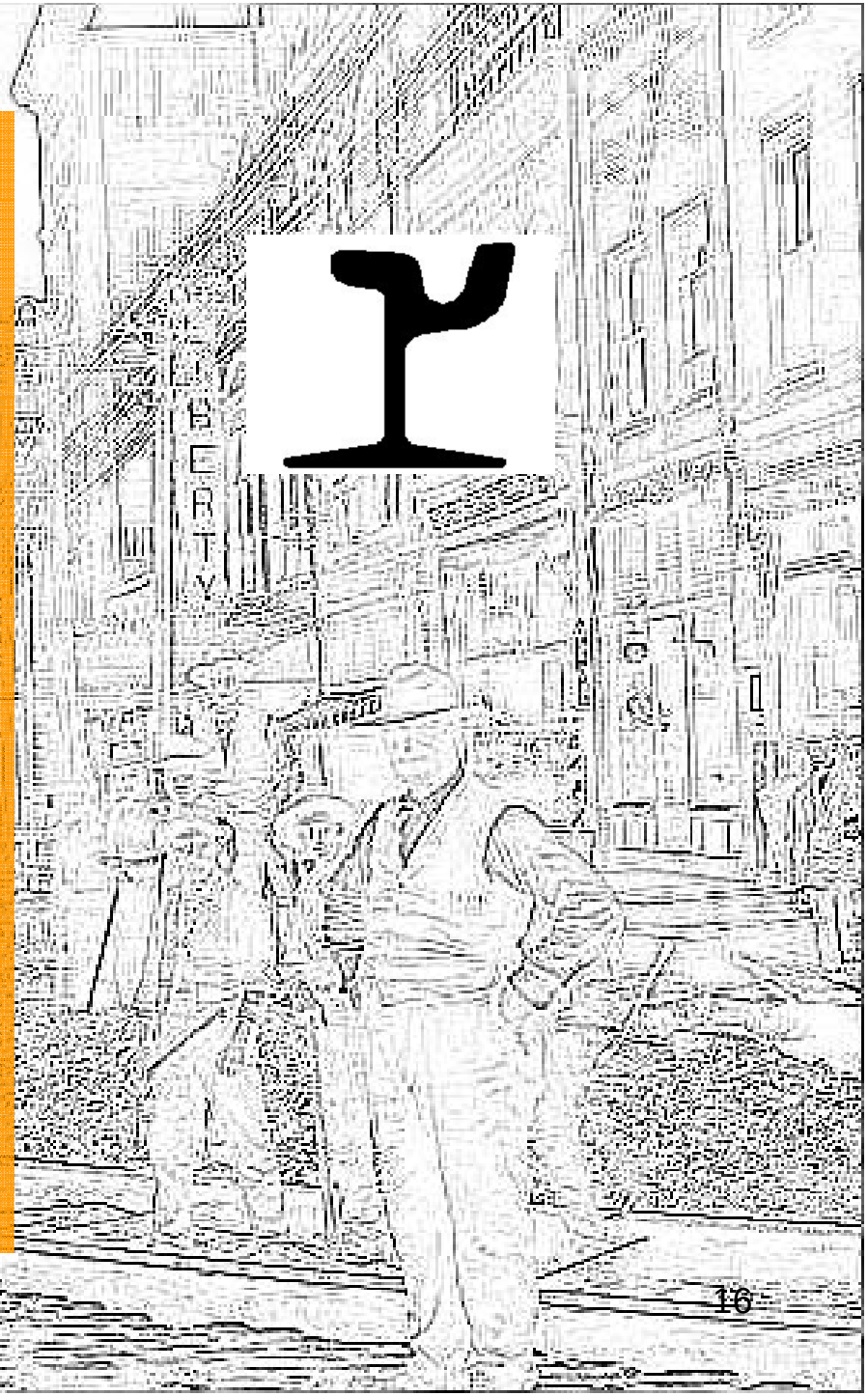

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