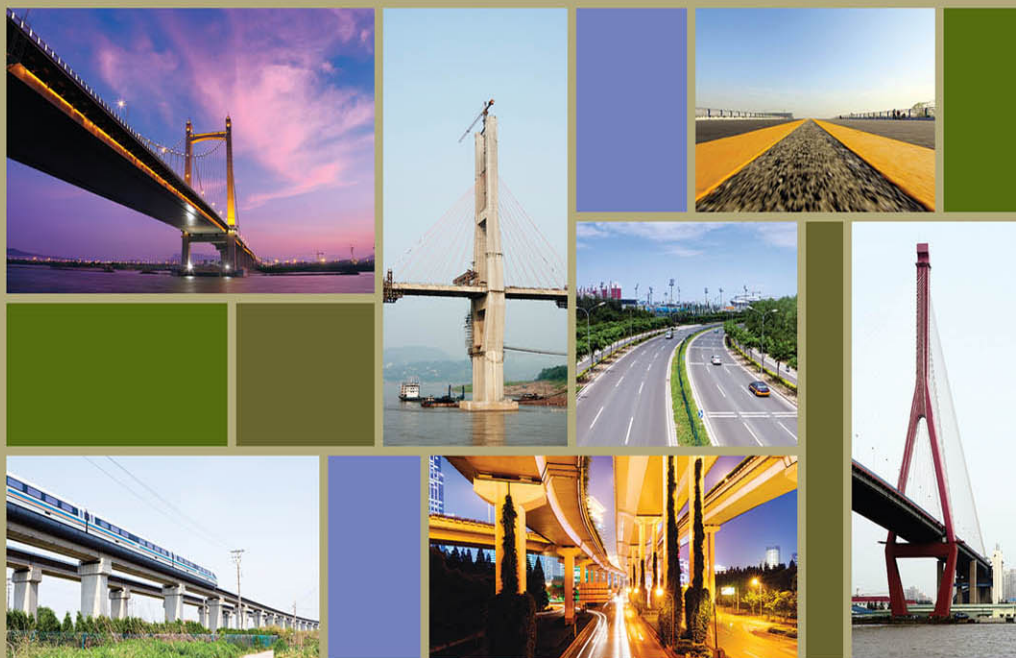




Challenges and Advances in Sustainable Transportation Systems

PLAN, DESIGN, BUILD, MANAGE, AND MAINTAIN



Edited by

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& DEVELOPMENT
INSTITUTE

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DEVELOPMENT CONFERENCE

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May 25–27, 2014

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Preface

Welcome to the 10th Asia Pacific Transportation Development Conference, organized by the International Chinese Transportation Professionals Association (ICTPA) and the Beijing Collaborative Innovation Center for Metropolitan Transportation. The conference is hosted by the Beijing University of Technology, Beijing Jiaotong University, Tsinghua University, and the Beijing University of Civil Engineering & Architecture. The ICTPA, the Beijing University of Technology, and the Transportation & Development Institute of the American Society of Civil Engineers (ASCE) are sponsors of the conference. The theme of the conference is “Challenges and Advances in Sustainable Transportation Systems: Plan, Design, Build, Manage, and Maintain.”

Transportation professionals are facing continuous challenges, including endless traffic congestion, ever-worsening environmental impacts, rising energy costs, and deteriorating quality of life. To meet these challenges requires new innovations to develop sustainable transportation systems through planning, design, construction, management, and maintenance. This special publication, produced by ASCE, is one of several official proceedings of the conference. This publication includes 86 peer-reviewed papers addressing 12 major areas: 1) Sustainable Multi-Modal Transportation Systems, 2) Strategies for Sustainable Development, 3) Efficient City and Regional Transportation Planning and Land Use, 4) Low Carbon Emissions and Energy Savings in Transportation, 5) Transportation Infrastructure Design Criteria, 6) Highway Infrastructure, 7) Subway and Rail Transit Infrastructure Planning and Design, 8) Intelligent Transportation Systems, 9) Bridge and Tunnel Planning, Design, and Construction Technologies, 10) Bridge Durability, 11) Bridge Health Monitoring and Evaluation, and 12) Geotechnical Design and Construction. Recent advances in new strategies, new methods, and new technologies are presented to address topics relating to funding, planning, design, construction, maintenance, and management of transportation facilities, including aviation, ports, harbors, highways, bridges, tunnels, rapid transit, and railroads.

One of the conference organizers is the ICTPA, a 27-year-old professional organization that has conducted conferences throughout North America and Asia to examine local and global transportation development issues. With the rapid growth in the recent years, China has become one of the largest transportation infrastructure markets in the world. In this publication, many authors who studied transportation development in China showcase new ideas and approaches that have emerged from some of the world’s largest transportation projects in that country. In addition, practitioners and scholars from the United States and other developed countries and regions present latest advances in transportation system management and maintenance. This unique combination presented by a distinguished group of internationally recognized practitioners and scholars provides a wide range of stimulation and inspiration in solving global transportation problems.

We believe that these proceedings will be invaluable to transportation professionals in advancing their careers. We hope that you enjoy the conference and the City of Beijing, catch up with old friends and colleagues, and make new ones.

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The organizers would also like to acknowledge many people whose names are not included above who have contributed their knowledge, time, and energy to make this conference successful.

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I-15 Bus Rapid Transit In-Line Stations

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ABSTRACT

Two in-line Bus Rapid Transit (BRT) stations along State Route (SR-) 15 at El Cajon Boulevard and University Avenue in the Mid-City area are under design. The in-lines stations are within the median of the freeway at the freeway level. The project will construct northbound and southbound BRT lanes within the existing median with offset side platforms at University Avenue and El Cajon Boulevard and they are part of Interstate (I-) 15 BRT system. This paper will discuss the planning and design challenges during the project implementation.

INTRODUCTION

The I-15 Express Lanes system includes a four lane “expressway within a freeway” for 20 miles from SR 163 to SR 78. A movable barrier in the middle of the expressway allows California Department of Transportation (Caltrans) to adjust the number of lanes in each direction based on traffic demand. The configuration can be two lanes in each direction, or three/one lane configuration. During special events, incidents or fire storms, Caltrans is able to change the lane configuration to accommodate the traffic need. There are nine direct access points to and from the general purpose lanes in each direction (approximately every two miles) and four direct access ramps (with a fifth planned for 2014) from transit stations along I-15. Right now, a new high-frequency BRT system to downtown is operating in the Express Lanes. There are four off-line BRT stations along I-15 between SR 56 and SR 78 in the north county. Another off-line BRT station is being built south of Mira Mesa Boulevard. South of I-8, I-15 becomes State Route 15. Two in-line BRT stations and BRT lanes along SR 15 between I-805 and I-8 are under design. These in-line BRT stations are proposed at University Avenue and El Cajon Boulevard along with BRT lanes to and from the BRT stations. The addition of these transit stations would be part of the I-15 BRT system. The purpose of the in-line BRT station project is to improve transit service and operations along the Mid-City portion of SR 15 in conjunction with local transit operations.

PLANNING PROCESS AND CHALLENGES

A. Project History

In the late 1990s, the segment of SR 15 between I-805 and I-8 (known as the 40th Street Corridor) was upgraded from an arterial to a freeway. As part of the implementation of the SR 15 segment between I-805 and I-8, a series of commitments and mitigations were developed between Caltrans, the City of San Diego, and other agencies. Early plans called for light rail transit to be built in the corridor, but as the region's transportation plans were refined, the mode of service in the corridor was designated BRT. A median-running rapid transit system was assumed in the design of SR 15 in the Mid-City area. The El Cajon Boulevard and University Avenue Plaza bridges were designed and constructed to enable vertical connections to future median BRT stations at the freeway level and provide room for commercial or retail uses on the bridge decks.

B. Community Interaction

San Diego Association of Governments (SANDAG) and Caltrans initiated a community-based planning process to determine the most effective location and design of the facility within the freeway Right of Way (ROW). A working group representing local communities and responsible transportation agencies has been closely involved with the development of the proposed BRT alternatives. This working group specifically comprised of SANDAG, Caltrans, the City of San Diego, MTS, and community stakeholders began to meet in late 2007 (and continued throughout 2008) to discuss the BRT alignment and station design concepts for service on SR 15 in the Mid-City area. Agency consultation and public participation for this project have been accomplished through a variety of formal and informal methods, including: project development team meetings, interagency coordination meetings, and public open houses. Initial meetings focused on identifying information needs, discussing community values, and developing screening criteria to be applied to assessing BRT station and alignment alternatives. In later meetings, the group worked to refine criteria measures, review the alternatives, and develop Context Sensitive Solutions (CSS). This process included two community workshops and 16 committee meetings through 2008 and 2009.

C. Alternatives Considered

Twenty Build Alternatives were considered during the project development process. In developing an initial range of transit facility alternatives, the PDT faced several challenges with the existing conditions in the corridor, including elevation and slope issues at the far northern and southern ends of the corridor, constrained ROW throughout the corridor, and operational concerns involving median-based stations and bus lanes. Key opportunities and constraints are summarized below:

- Constrained Corridor Cross Section Width:

SR 15 in Mid-City is depressed below the surrounding urban streets, bounded by slopes and retaining walls. The median width ranges from approximately 30 feet to 64 feet and would accommodate standard BRT lanes and shoulders. There is only very limited room for platforms and pedestrian access within the median. Outside widening is not an option due to high retaining walls and adjacent local communities. Additionally, there is insufficient room for both continuous HOV lanes and BRT lanes with bus bypass within the median. MTS requires bus bypass lane in case buses break down at the platform area.

- **BRT Vehicle Right-Side Doors:**

The constrained width of the corridor limits design options for stations. Using buses with doors on the left side of the vehicle could maximize the use of the station area with a center platform. After discussions with MTS staff and the community, it was determined that stations should be designed for right-sided loading. The primary reasons were the inability to use left-side door buses elsewhere in the system and the creation of a requirement to dedicate vehicles exclusively to the bus lane. Buses with doors on both sides were considered, but they were not pursued due to the reduction in available seating.

- **Median BRT At-Grade Crossovers:**

Median BRT at-grade crossovers have been used in other parts of the state. This approach would allow buses to cross over at-grade lanes to run on the left side of the median into a station, and have right side doors open onto a platform. However, Caltrans and the California Highway Patrol (CHP) headquarters determined in 2007 that median in-line stations with a contraflow BRT operation (buses traveling in the opposite direction of general purpose lane traffic) utilizing at-grade bus crossings are not acceptable due to safety concerns associated with the at-grade crossings.

- **Grade Separated Crossovers:**

Providing a grade separated crossover structure at the two ends of the bus lane presents an opportunity to provide contra-flow operations for right-side BRT doors and would mitigate some of the potential safety hazards from errant vehicles.

The PDT developed a series of alternatives designed to provide BRT transit service within the corridor. The range of these initial alternatives can be grouped into four categories:

- 1) **Median-Based** - Service would run in the freeway median, either at-grade, underground via a tunnel, or above-grade through elevated lanes.
- 2) **Shoulder-Based** - These alternatives would make use of shoulder-based stations, and could either operate in mixed-flow lanes or along freeway shoulders.
- 3) **Ramp-Based** - The ramp-based alternatives would be the closest equivalent to the current transit service along this section of SR-15, but the proposed alternatives would involve relocating existing stops from their current nearside location on the freeway off-ramps at El Cajon Boulevard and University Avenue to the far-side on-ramps.
- 4) **Elevated Hybrid** - These unique alternatives include a separate transit way running the length of the corridor along an elevated structure running either within or adjacent to the freeway right-of-way. Certain alternatives also include multi-modal elements, such as bicycling or walking trails.

D. Viable Alternatives

This section includes the build alternatives that were developed by the PDT to achieve the project purpose and need, while avoiding or minimizing environmental impacts. The viable build alternatives described and evaluated include Alternative 1: Median Bus Lane with Center Platforms; Alternative 2: Median Bus Lane with Side Platforms; and Alternative 4: Shoulder Bus Lane with Ramp Stations. Note that Alternative 3 with outside shoulder bus lane and outside shoulder stations was rejected during the environmental phase due to safety concerns.

1) Alternative 1 (Median Bus Lane with Center Platform Stations)

Alternative 1 would construct northbound and southbound dedicated 11-foot BRT lanes within the existing median. It would include contraflow bus traffic (buses traveling in the opposite direction of general purpose lane traffic) along, and between, the two BRT stations that would be separated from general vehicle traffic by a concrete barrier. The BRT stations would be shielded from the adjacent general purpose lanes. This alternative does not include at-grade crossings as shown in Figure 1.

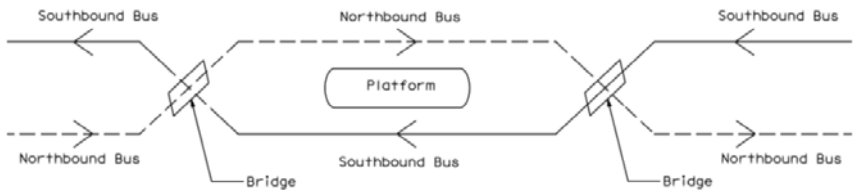


Figure 1. Alternative 1 Conceptual Drawing

With a center platform design, two crossover bridges would be constructed to support contraflow operations. The Landis Street pedestrian bridge has to be relocated. The southbound BRT lane would cross over the northbound BRT lane south of Adams Avenue. Center platform stations would be located at University Avenue and El Cajon Boulevard only. The northbound and southbound platforms at these two locations would be approximately 140 feet in length and 16-18 feet in width and be connected under the overcrossing. The bus platforms will be accessed by patrons through pedestrian bridges, stairs and elevators from the existing overcrossings of University Avenue and El Cajon Boulevard. Other station amenities include shelter canopies, ticket vending machines, variable message signs, blue light emergency phone and CCTV cameras. A fourth leg pedestrian crossing would be established across University Avenue and El Cajon Boulevard at the ramp intersections with northbound and southbound SR 15.

2) Alternative 2 (Median Bus Lane with Side Platform Stations)

Alternative 2 would not include contraflow bus traffic since separate northbound and southbound BRT stations would be positioned to the right of the bus lane within the median at both University Avenue and El Cajon Boulevard locations; therefore, no new crossover bridge construction would be required. The BRT stations would be separated from general purpose lane traffic by a concrete barrier. The BRT stations would also be shielded from the adjacent general purpose lanes. The northbound and southbound side platforms at both University Avenue and El Cajon Boulevard would each be approximately 140 feet in length and 12-14 feet in width. However, the side platforms cannot be placed at same location because of the median width constraints. Since this alternative utilizes offset platforms, the platform width could be maximized without compromising bus bypass given the limited mediana width as shown in Figure 2. Similar to Alternative 1, this alternative will also include BRT lanes and station amenities.

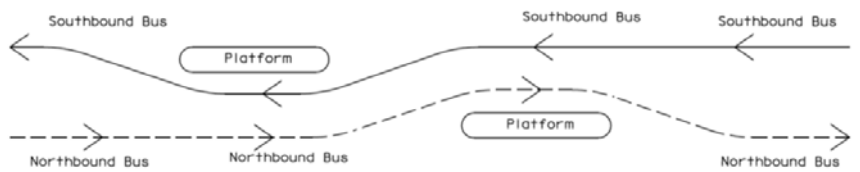


Figure 2. Alternative 2 Conceptual Drawing

3) Alternative 4 (Shoulder Bus Lane with Ramp Stations)

Alternative 4 would provide BRT shoulder stations on the outside of the northbound and southbound on-ramps at University Avenue, El Cajon Boulevard, and Adams Avenue as shown in the Figure 3.

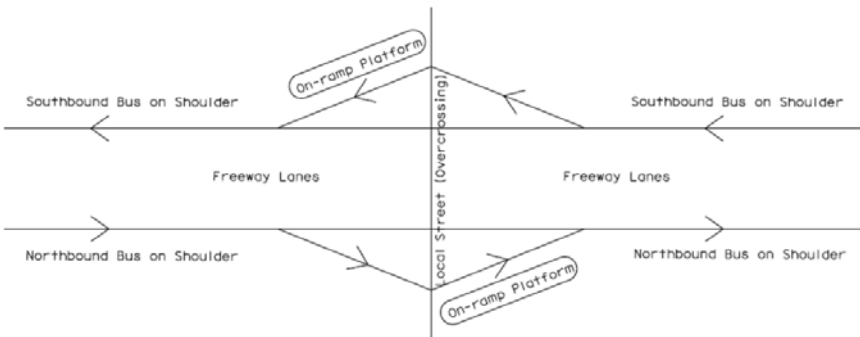


Figure 3. Alternative 3 Conceptual Drawing

Buses would travel in the general purpose lanes and utilize the outside shoulders during peak traffic hours with congestion. The existing bus stops in the middle of the off-ramps at El Cajon Boulevard and University Avenue will be removed and replaced with a bus priority lane for buses to cross the intersections. A dedicated BRT lane would be located on the on-ramps to allow the buses to enter and exit the station areas. Ramp meters would create a queue jump to allow buses to merge with general traffic, and these BRT shoulder stations would not be separated from general vehicle traffic by a concrete barrier.

4) No Build Alternative

Under the No Build Alternative, no BRT stations would be constructed in the project corridor, and BRT lanes would not be constructed as part of the current project. However, the no build alternative does not meet the project purpose and need.

5) Transportation System Management/Transportation Demand Management Alternative

Transportation Systems Management (TSM)/Transportation Demand Management (TDM) measures are strategies to enhance the efficiency of the transportation system at a lower cost. TSM measures seek to increase the number of vehicle trips that can be carried without adding lanes. TDM focuses on regional strategies for reducing vehicle trips and miles traveled, and increasing vehicle occupancy. Many of these measures are already incorporated or retained in the project alternatives. TSM measures include modifications to ramp meters and auxiliary lanes, and managed lane implementation via the regional HOV system. For the TDM strategies, ridesharing, multi-modal use, and transit strategies are also a part of the project. Because of the overlap of these strategies, a separate TSM/TDM alternative was not evaluated.

E. Environmental Issues

1) Mitigated Negative Declaration/Findings of No Significant Impacts

A Final Initial Study with Mitigated Negative Declaration and Environmental Assessment with Finding of No Significant Impact (Final IS/EA FONSI) has been prepared in accordance with the California Environmental Quality Act (CEQA), National Environmental Policy Act (NEPA) and Caltrans' environmental procedures, as well as all applicable State and federal environmental regulations.

2) Parks and Recreational Facilities, Section 4(f)

No long-term effects to parks or other recreational facilities would result from implementation of the preferred alternative and no use of 4(f) resources would occur. Several parks and recreational facilities located within 0.5 mile of the project are eligible for protection under Section 4(f). However, the proposed project would not

result in a Section 4(f) “use” of any eligible properties and proximity impacts would not result in a constructive use of any eligible properties.

3) Visual Considerations

Visual mitigation will be proposed to the preferred alternative and they fall into the following categories: 1) wall treatments; 2) elevator or architectural treatments and; 3) landscape replantings. Most of the project impacts will require mitigations from all three of the above categories. Project features associated with Alternative 2 will only moderately contrast with the unity and visual organization of the space. Alternative 2 would result in a moderate impact to the visual quality. The proposed project has little to no effect on the existing architectural features that are of importance to the project site and the setting may benefit from the project by increasing the visual organization and vividness.

4) Paleontological Monitoring

A paleontological monitor will be onsite on a full-time basis during the original cutting of previously undisturbed deposits of high sensitivity formations to inspect exposures for contained fossils. The paleontological monitoring is required for the large-diameter borehole for columns to support the elevated pedestrian bridges and for the elevator shafts in the preferred alternative. In the event that fossils are discovered the Caltrans will take the correct actions under the paleontological monitor’s guidance to address the issue accordingly.

RECOMMENDATION

Local agencies have been consulted with respect to the recommended alternatives and their views have been considered. Alternative 2 emerged as the preferred alternative. This is due to level of visual impact, comparative construction costs, community input, and ROW factors. Alternative 2 has much less visual impacts since it has no BRT crossover bridges compared to Alternative 1. It does not require contraflow operation of buses that will travel in an opposite direction of general purpose traffic in Alternative 1. It also does not require Landis Street Pedestrian Overcrossing relocation as required in Alternative 1. The construction cost of Alternative 2 is much less than that of Alternative 1 but it provides same level of service (LOS) to the BRT users. Community responses were supportive of the median bus lane alternatives. Compared to Alternative 4 this preferred alternative has no ROW impact and less utility impacts but it will provide center running BRT service that is faster than that of Alternative 4. Overall, Alternative 2 is cost effective and will satisfy project goals with minimal environmental and right of way impacts.

CONCLUSION

The BRT station project was planned and studied per CEQA, NEPA and Caltrans Project Development Procedures Manual (PDPM) requirement. The offset side

platform concept of Alternative 2 was a creative solution given the constraints and challenges. The public outreach and the involvement of the community group were successfully implemented during the project initiation and environmental study phase. Caltrans and SANDAG are using this project as a public outreach model for future BRT station projects.

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Advanced Green Port and Waterway Development through Integrated, Energy-Efficient ITS/ICT Deployment in China and Asia

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ABSTRACT

This paper summarizes the comprehensive system development and intermodal traffic management center deployment currently underway in China that can serve as an excellent development model for other cities in China and Asia. ShenZhen, China, has been actively expanding its port infrastructure and urban development with a unique sustainable, environmentally-friendly enhancement. The ShenZhen Transport Commission in ShenZhen City is currently integrating planning, system design, prototype demonstration, logistics, and eco-friendly green port operations and water transport to improve the environment, resources, community, and investment to successfully attract new energy-efficient industry through integrated high-tech development and improved intermodal logistics. As one of China's major gateways with a unique economic development zone, ShenZhen is applying various ITS technologies to reduce emissions, improve energy efficiency, and provide green logistics to link Hong Kong and the Chinese Mainland for a sustainable inland waterway environment, integrated logistics, and system planning development.

INTRODUCTION

China's ongoing Twelfth National Five-Year Development Plan has identified a variety of areas and new initiatives for sustainable energy solutions and economical, environmentally-friendly development plans for 2011–2016. Among them, China's key industrial sectors in transportation, construction, business, and non-government will focus on efficient energy use and pollution reduction to improve the country's energy savings and sustainable environment. China currently has three of the world's top 10 largest container ports, handling 34% of global container transportation, and many other ports and waterways are also expanding rapidly. This paper summarizes ShenZhen Port's current Intelligent Transportation Systems (ITS) development and Information Communication Technology (ICT) deployment to achieve desirable, integrated energy efficient operations similar to the Green Korea Fukuyama Port (Busan, Korea), the Long Beach Green Port (USA), and the Los Angeles Green Port (USA).

ShenZhen, China, has been actively expanding port development with a unique sustainable, environmentally-friendly enhancement. The ShenZhen Transport Commission is integrating planning, system design, prototype demonstration, logistics, and eco-friendly green port operations and water transport development to improve the environment, resources, community, and investment to successfully

attract new energy-efficient industry through integrated high-tech development and improved intermodal logistics. This integrated planning, system design, and system deployment will include prototype demonstrations with measurable energy savings and environmental improvements and will support the ShenZhen New Port development among five ports in the Pearl River Delta. As one of China’s major shipping ports with a unique economic development zone, ShenZhen will apply various ITS technologies to reduce emissions, improve energy efficiency, and provide green logistics to link Hong Kong and the Chinese Mainland for a sustainable inland waterway environment, integrated logistics, and systematic planning development. This paper summarizes the comprehensive system development and intermodal traffic management center deployment that are currently underway in China, which can serve as an excellent development model for other cities in China and in Asia.

During the Eleventh Five-Year Plan, the ShenZhen municipal government put enormous efforts into ITS construction and formed the basis for an ITS environment primarily in the following areas as a part of the overall, national Internet-of-Things or Logistics, as shown in Figure 1:

- 1. Preliminary formation of an urban ITS environment.
- 2. Acceleration of ITS infrastructure construction.
- 3. Continued promotion of a top-level application system.
- 4. Pilot implementation of an urban traffic information center.
- 5. Joint development of ITS and transport information.

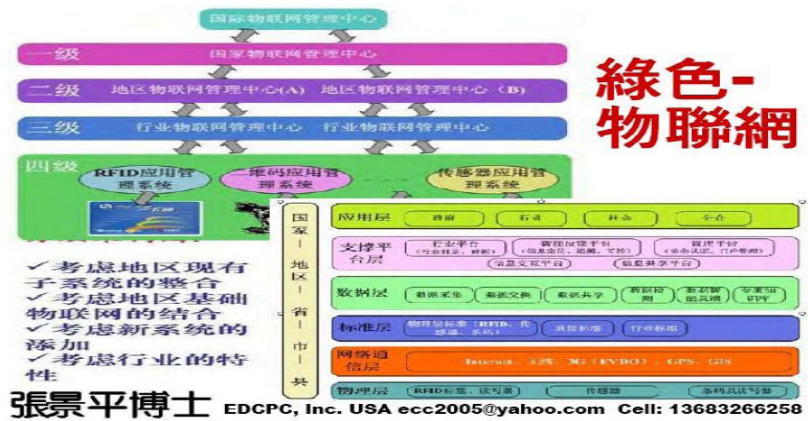


Figure 1. China’s National Internet-of-Things or Logistics Efforts.

DEVELOPMENT METHODOLOGY

To support full-scale ITS system deployment, ShenZhen ITS is applying the methodology widely used in U.S. regional and urban ITS architecture planning that uses (1) market package-based ITS planning, (2) demand-oriented planning, and (3) traceable ITS planning approaches.

The approach starts with analyzing the system inventory, clarifying the ITS user needs, and setting planning objectives, thereby setting the necessary national ITS architecture market package as the entry point. It clarifies the deployable, practical ITS market package that can achieve goals, then develops the regional and urban ITS architecture and performs a needs analysis of the ITS implementation standards. The implementation project and deployment sequence will be formulated to match the corresponding deployment. Finally, an operations and maintenance program will be developed, including a system update to safeguard programs based on the established regional and urban ITS architecture.

The planning approach will be developed similar to the market package-based approach to move the basic planning process forward. However, the intended process will clarify the regional and urban ITS planning vision and goals to closely model the ITS demand analysis for a practical, realistic system implementation. This is a manifestation of concept planning, and the content and ideas of the planning are similar to the above two approaches. During the planning process of this approach, any given planning objective will be reflected in the following customer service, architecture, and deployment.

Compared with the ITS planning used before, this ShenZhen ITS planning approach focuses more on organizational needs and a planning horizon according to the characteristics of ShenZhen city. During the planning process, the U.S. ITS architecture is referenced with more focus on the needed localized market package and equipment package for ShenZhen's long-term development needs, as shown in Figure 1. ShenZhen's demand-based, integrated regional and urban ITS planning methodology provides a clear, concise, and effective vision of the socio-economic and transport development needs. It analyzes ShenZhen City from multiple perspectives on ITS needs and proposes the ITS development. The process uses China's national ITS framework as the base, developing the general ITS framework for Shenzhen, determining the comprehensive, logical, structured blueprint, and developing specific ITS applications, user services, and functional building objectives to support ITS implementation.

IMPLEMENTATION APPROACH

ShenZhen's ITS Twelfth Five-Year Plan seeks to strengthen ShenZhen as an international city, coordinate ShenZhen's regional and urban objectives, and support the "green port" logistics hub development for integrated, seamless transport for all people, goods, and service. The overall implementation guideline is to develop a "people-oriented, customer-centered, public service that can better support international industry management and governmental interagency decision-making." ShenZhen will build a "sustainable development world class city" that can optimize infrastructure investment, relieve traffic congestion, facilitate travel, and minimize energy savings with special attention to achieve (1) integrated ITS development for safe, efficient, and seamless transport operations and (2) regional transport integration of ShenZhen City and other cities on the east coast of the China Pearl River Delta, including the Hong Kong, Dongguan, and Huizhou Special Economic Development Districts, as shown in Figure 2.



Figure 2. ShenZhen information and communication technology approach.

The integrated ITS development will promote a convenient, effective, comfortable, and safe urban transport environment to achieve a green transport structure and build a new transport information service architecture to secure urban transportation efficiency and safety, improve the utilization of transport infrastructure, and promote harmony and interoperability between humans and roads and humans and vehicles. ShenZhen's ITS development will focus on four development areas: as ShenZhen steps into the "Internet-of-Things," its municipal government is carrying out "Smart ShenZhen" overall planning as the basis for "Smart Industry" and "Smart City." The intelligence industry is facing a perception system, network system, platform system, and application system. The "intelligent city" is facing government, industry, and public.

The "advanced stage development" will be divided into three stages. The first stage (2011–2012) will form the basis, the second stage (2013–2014) will focus on emphasized systems and performance, and the third stage (2014–2015) will be re-creative. The ITS construction in ShenZhen started late, but it has a higher starting point. Based on the summary and learning from the experience of advanced cities, it plans to create a new era in ITS during its Twelfth Five-Year Plan.

The Integrated e-Travel Platform Support Development ITS information platform is a supportive part of the ITS development. During this five years, it will build three sub-centers: a traffic control command center, a traffic operations command center, and an urban transport information center. The idea of a traffic information service that focuses on public convenience, efficiency, comfort, and safety is the core of the ITS system nodes. It is based on the urban transport basic information environment, using modes such as the Internet, cell phones, VMS, mobile TV, and onboard navigators as the carriers. It will form a transport information network.

As part of the customer-service-oriented decision supports supporting a global logistics hub and an international transit metropolis, the focus should be on providing traffic information service to travelers and publishing real-time dynamic urban traffic information, and, at the same time, providing government, industry, and business

traffic decision support. During the full-scale ITS construction and implementation process, the market mechanism will be used to play its role, take advantage of public/private partnerships for added potential investment, and promote the rapid development of the traffic information service.

KEY TASKS FOR SUCCESSFUL DEVELOPMENT

ShenZhen ITS development should be adapted to industrialization and urbanization, effectively satisfy urban transport needs, and achieve sustainable development. By building a modern integrated transport system during the next five years, ShenZhen will develop rail transport as the backbone of a public transport-oriented transport mode and construct a developed transportation network of roads and rail. The technical roadmap includes constructing a transit metropolis, creating an effective transport system, and building urban transport infrastructure that can integrate highway, rail transport, waterway, air transport, and international seaports. The new ShenZhen ITS construction includes the “1+6 System Concept,” i.e., one platform and six systems, including a Traffic Information Exchange Platform and (1) an Integrated Traffic Monitoring System and Road Traffic Control System, (2) a Transport Management System, (3) a Traffic Operation and Command System, (4) a Public Transportation Information Service System, (5) a Traffic Management and Emergency Management System, and (6) a Simulation Decision Support System.

Through technical and institutional integration, ShenZhen ITS information platform construction will achieve a reasonable full sharing of information and resources and will provide decision support for the urban transport system that can integrate the service needs from three sub-centers: (1) a traffic control command center, which focuses on traffic safety and traffic order monitoring, (2) a traffic operations command center, which focuses on urban transport operation analysis, and (3) an urban transport information center, which focuses on the traffic information gathering, exchange, analysis, processing, and sharing and integrating existing platforms, including an urban spatial information platform and an urban traffic simulation platform from the Planning Bureau, a GPS monitoring and managing platform from the Transportation Bureau, and a traffic control platform from the Traffic Police Bureau. The integrated system will create a unified interface of urban traffic information sources that integrates data sharing among various municipal operating agencies and departments across nine different traffic management systems, as shown in Figure 3:

- 1) Trip distribution network – uses travelers as research objects and gathers and analyzes traffic generation and attraction, travel status and location distribution, travel mode choosing, traffic dynamic assignment.

- 2) Road traffic network – contains four levels of the road network: the expressway on the east coast of the Pearl River Estuary, the urban road network, the original SAR road network, and the urban CBD network with historical and real-time dynamic information.

- 3) Bus line network – publishes real-time passenger volume status, bus line congestion, and estimated time of arrival.

- 4) Rail passenger network – publishes rail station passenger numbers,

interchange information of each rail station and bus stop at 5-, 10-, and 15-minute level isochrones and important municipal facilities around each exit of rail stations.

5) Signal control network – accesses intersection location information and forms an electronic sand table to supply signal control information and management.

6) Parking guidance network – includes three levels of guidance: area, arterial, and residential; gathers information on parking lots in real-time and publishes parking locations via VMS, cell phone, or on-board navigator.

7) On-call taxi network – includes a taxi on-call system, which gathers and processes taxi GPS data and supplies on-call service.

8) Hub transfer network – includes information of the hub structure, location of rail stations, bus stops and taxi stops, entry and exit schedules, transfers, and municipal facilities.

9) T-GIS system – includes basic geographic information of urban infrastructure.



Figure 3. Integrated ShenZhen ICT Network.

ShenZhen’s ITS information platform will strengthen the capability of regional and urban road control, and the transport service will be greatly enhanced to serve the general public. It will complete a unified system design and integration framework through an integrated urban transport and dispatching system for improved bus information to attract people using low energy consumption, low pollution, low cost, and multimodal travel. The user services include an ITS information platform, government decision support, public transport management, traffic control and management, traffic coordination and emergency command, road infrastructure management, traffic information service, ETC, traffic safety, and vehicle safety through 47 user services and 154 user sub-services, such as constructing ITS based on the integration of the highway network and hub, building ETC systems, achieving highway operation monitoring, and publishing information. It will form independent systems such as TMIS, DMIS, ATIS, and a ticket booking and selling system, a package management system, etc., and will accelerate the construction of waterway transport information infrastructure and strengthen the development, application, and integration of a waterway transport information source.

It will focus on systems including integrated port information management system, intelligent container management and scheduling system, Port facilities condition monitoring diagnosis maintenance system, EDI, etc. It will integrate the content of airlines, airport companies, military control, and aviation weather. It will focus on systems such as ATC, ATFM, ASM, and CNS/ATM and will achieve the cross-border traffic integration of the Shenzhen-Hong Kong Corridor, especially cross-border traffic information service integration. It will build a cross-border traffic information service environment, such as VMS of cross-border, dynamic port interchange information service, etc.

CONCLUSION

ShenZhen's ITS development, initiated in 2004, will implement various real-time dynamic analysis, research, and a traffic evaluation system to drastically improve urban transport planning and construction/management and will reengineer the traffic management process. A corresponding "nine integration" has been proposed to integrate the regional and urban ITS architecture, a ShenZhen and Hong Kong cross-boundary traffic information service, an ITS-capable modern transport system, Special Economic Zones ITS construction, ShenZhen Great bus service, ITS architecture and Internet-of-Things, ITS construction and management, human vehicle road and environment, and ITS and transport information service. Once completed and totally in operation, ShenZhen's comprehensive ITS development model will be able to seamlessly integrate planning, system design, and demonstration of the eco-friendly greening of port terminals and water transport to improve the environment, resources, community, and investment to successfully attract new transport energy-efficient industry to nearby high-tech industrial development.

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China's Challenges on Airport Land Use Compatibility Planning

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ABSTRACT

One of the challenges facing airport development is the ongoing encroachment of incompatible land uses around airports such as high intensity residential areas, schools, and environmentally sensitive land uses. In California, airport land use compatibility issues are particularly challenging since airport operators generally do not have direct authority over local land use decisions. In addition, local governments often consider economic incentives from land use development around airports outweigh airport land use compatibility considerations. China is building 50 to 60 airports in the next five years, including the world's largest airport, the Beijing Daxing International Airport by 2018. Using various U.S. examples, this paper focuses the airport land use compatibility planning issues China may be facing. A right balance through the use of airport land use compatibility planning must be sought between promoting the economic benefits and preserving the vitality of new airport development versus protecting the safety and the environment of nearby communities.

INTRODUCTION

One of the ongoing challenges facing airport development is the issue of airport land use compatibility planning. Airport officials and local governments have been grappling with airport land use compatibility concerns for decades. Ongoing encroachment around airports such as high intensity residential areas, schools, and environmentally sensitive land uses, etc. is a major issue for governments, airport operators, and aviation communities around the world. Encroachment from land use development diminishes the necessary safety buffer between an airport and the communities they serve. Tall structures built too close to airport safety zones and environmental concerns such as aircraft noise and carbon emissions often place significant constraints on airport operation and expansion.

This paper examines various airport land use compatibility issues in the U.S. References were made from the Santa Barbara Municipal Airport in California and the Denver Stapleton International Airport in Colorado as they relate to the building of the new Beijing Daxing International Airport in China which is scheduled for completion by 2018.

ALUCS IN CALIFORNIA

In the U.S. guidelines of airport land use compatibility planning are provided under federal, state and local levels. In California, Airport Land Use Commissions (ALUCs) are mandated by state law to develop and implement an Airport Land Use Plan (ALUP) for each public-use airport. The ALUC is a statutorily created public administrative agency responsible for conducting evaluations on land use development projects in the vicinity of airports for consistency with the locally adopted ALUP. The purpose of the state law is "...to protect public health, safety, and welfare by ensuring the orderly expansion of airports, and the adoption of land use measures that minimize the public's exposure to excessive noise and safety hazards..." (California Public Utilities Code, 2012).

AIRPORT LAND USE COMPATIBILITY PLANNING ISSUES

In general, issues of airport land use compatibility planning can be dividing the following four major areas of concern (Caltrans, 2011):

- Airport Safety
- Aircraft Noise
- Airspace Protection
- Overflight

Figure 1 depicts a 3D conceptual representation of the four areas of concern around airports (Santa Barbara County Association of Governments, 2012).

AIRPORT SAFETY

Airport safety refers to the protection of people on the ground and in the air from aircraft accidents. Because aircraft accidents do occur with unpredictable consequences, the concept of risk is crucial in the assessment of airport safety and compatibility. Two variables are generally used to determine airport safety: 1) the degree of risk and frequency posed by potential aircraft accidents occurring in the vicinity of airports, and 2) the consequences of such accidents affecting the safety of the community.

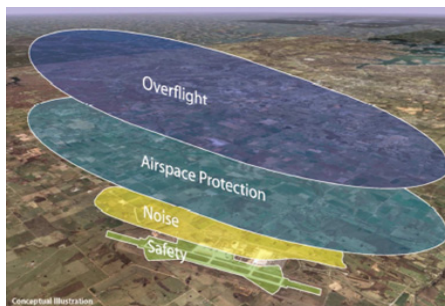


Figure 1. A 3D Conceptual Representation of Airport Land Use Compatibility

In the U.S., airport safety standards and guidance are set forth by the Federal Aviation Administration (FAA) Advisory Circular, Airport Design, AC150/5300-13. These standards and guidance are limited to the immediate vicinity of runways, the runway protection zone at each end of runways, and the protection of navigable airspace. These areas normally encompass within the airport boundary. More comprehensive policies and guidelines for airport land use compatibility planning are available from each state, for example, the California Airport Land Use Handbook developed by the Department of Aeronautics for the State of California. Their roles are for the safety of aircraft operations, leveraging for promoting compatible land use planning through grant assurances.

In California, six generic safety zones were developed providing information on various shapes and dimensions of these zones. They were established to reflect the relative risk levels of aircraft accidents based upon historical accident data relative to a runway end.

- *Zone 1: Runway Protection Zone*
- *Zone 2: Inner Approach/Departure Zone*
- *Zone 3: Inner Turning Zone*
- *Zone 4: Outer Approach/Departure zone*
- *Zone 5: Sideline Zone, and*
- *Zone 6: Traffic Pattern Zone.*

Most local ALUCs further developed similar safety zones and land use compatibility policies for each public-use airport under their jurisdiction based on local conditions. Figure 2 is an example of safety zones developed for the Santa Barbara Municipal Airport (Santa Barbara County Association of Governments, 2012). Airport safety compatibility criteria and policies for each zone were established in terms of population density (persons per acre) and intensity of land uses (dwelling units per acre). Urban or rural compatibility criteria were developed to determine whether land use development projects are “compatible,” “conditionally compatible,” or “incompatible” with the ALUP.

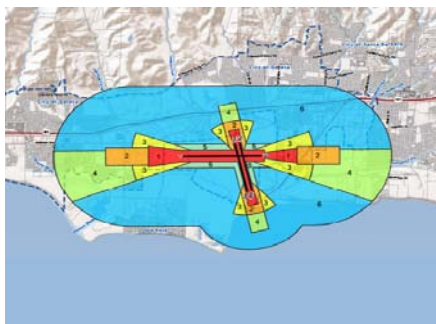


Figure 2. Santa Barbara Municipal Airport Safety Zones (Preliminary)

China is building the new Beijing Daxing International Airport, the world’s biggest airport, by 2018. It is located in the Daxing District to the south of the Beijing capital city (Figure 3). Part of the ongoing Daxing Airport Master Plan calls

for the planning of 2,680 hectares (6,620 acres) of land for an initial planning of six runways with two additional runways in the future for an ultimate capacity of 130 million passengers. The site is located on populated suburban residential areas. On the outset, just the sheer number of runways would involve complex safety zones to be developed in order to minimize the risk of potential aircraft accidents in and around areas of each runway end. Thus, at the initial stage, maintaining a continued communication to gain understanding and acceptance from various airport stakeholders and local communities regarding the need, extent, and potential impacts of this new airport project would be of utmost priority. One of the effective approaches would be to adopt a “structured” Memorandum of Understanding (MOU) as part of the initial planning process for effective implementation of airport safety zones.

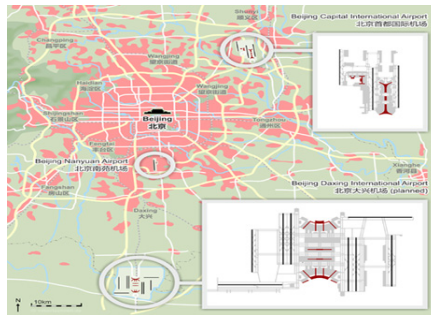


Figure 3. Beijing Daxing International Airport and Vicinity

AIRPORT NOISE

Aircraft noise has long been regarded as one of the health hazards for people living around airports. Prolong exposure to aircraft noise can be harmful to humans, from damage to hearing to development of cardiovascular disease (Santa Barbara County Association of Governments, 2012). In the U.S. comprehensive noise policies and regulations on aircraft noise and compatibility planning are provided by the federal and state governments. The FAA Federal Aviation Regulations (FAR) Part 150 Study is often employed by airport authorities as a tool to prescribe standards, procedures, and methodology governing the development, submission, and review of airport noise compatibility programs. These programs provide evaluation and approval of airport noise exposure maps, measurement of airport noise impacts and implementation of noise compatibility program strategies.

Figure 4 depicts the 2011 Noise Exposure Map (NEM) for the Denver International Airport (DIA). The noise impacts are expressed in terms of Day Night Sound Level contours (LDN) in decibel for an average day (Business FlyDenver, 2011). In general, residential land uses within the 65 LDN and above noise contour are considered incompatible. The core of the FAR Part 150 Study is to develop existing and future NEMs for evaluation of noise impacts for surrounding communities with recommendations for effective noise mitigation program strategies.

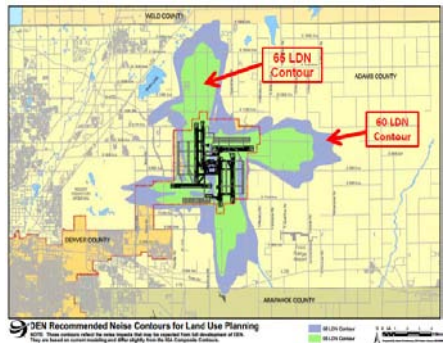


Figure 4. Denver International Airport 2011 Noise Exposure Map

The runway layout of DIA is somewhat similar to the new Beijing Daxing International Airport (Figure 5) (NACO, 2012). The shape and extent of the DIA NEM could, to certain extent, mimic a generic sample outlook of noise impacts for the new Beijing Daxing International Airport. The DIA NEM is borrowed here to illustrate the possible noise impacts and development of mitigation strategies for the new Daxing Airport noise compatibility planning.

While the DIA NEM could provide a glimpse of the shape of the potential noise impact for the Daxing International Airport, developing the actual NEM for the new airport will be challenging. For example, how much domestic air traffic could be effectively transferred from the current Beijing Capital International to Daxing International at its initial opening stage in order to relieve the urgently needed capacity at Beijing Capital, and how the air traffic would be distributed among the six initial runways at Daxing would need to be fully analyzed. Detailed noise analyses under various runway use scenarios would be needed in order to generate a comprehensive noise impact environment for the new airport. Noise matrices would need to be developed under “compatible”, “conditionally compatible”, or “incompatible” land use categories for existing and future land uses around the Daxing District. Furthermore, integration of military aircraft operation from Nanyuan Airport under short-, medium-, and long-term scenarios would need to be examined as well.

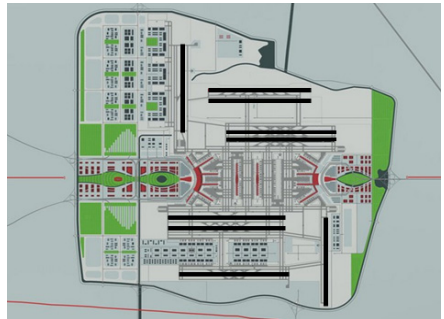


Figure 5. Daxing International Airport Master Plan

For discussion purposes, some of the noise mitigation options available to Chinese airport authorities could include:

- Curfew to limit certain aircraft operation during certain night hours;
- Preferential runway treatment for arrivals, departures and engine run-ups during certain operating conditions;
- Curfew on operational activities and restrictions on certain noisier aircraft types;
- Continued open dialogue for agreement with various airport stakeholders, local communities and the general public; and
- Real estate publicity and notifications such as “Notice of Airport in Vicinity” as early in the planning stage as possible, etc.

AIRSPACE PROTECTION

Any physical obstructions to the navigable airspace such as tall towers and land use structures that create height, visual, electronic or thermal interference with aircraft navigation and communication can be hazardous to flight. The purpose of airspace protection is to ensure that any tall structures and other land uses do not cause hazards to aircraft operation within the airport vicinity.

The FAA FAR Part 77 Airspace Plan establishes standards for determining obstructions to air navigation. Figure 6 depicts the Santa Barbara Municipal Airport Part 77 Airspace Plan. It shows all airport imaginary surfaces and specific information on approach and departure slopes, dimensions, height restrictions based on the type of approach available for each runway end (e.g., visual, non-precision, precision) at the Santa Barbara Municipal Airport. Part 77 also provides requirements for notice to the FAA of certain proposed construction for aeronautical studies to determine effects on the safe use of airspace. This includes tall structures, antennas, trees, temporary mobile objects such as cranes, etc.

In the U.S. the FAA has limited authority on matters related to local land uses. Thus airspace protection may warrant local ALUC review. This is particularly the case when proposed land uses were not already evaluated on the general or specific plan to be found for consistency with the ALUP. Additional guidance on local airspace protection may need to be considered as well. For example, electrical interference with navigational signals or radio communication between the airport and aircraft might create interruptions to pilots. Lighting may create glare to pilots. Smoke or other impairments may have impacts on visibility in the airport vicinity. Fish ponds may create wildlife hazards such as bird strikes, etc. The FAA specifically considers certain land uses such as waste disposal sites and landfills to be incompatible if these land uses were located within 10,000 feet of a runway used by turbine-powered aircraft or 5,000 feet of other runways.

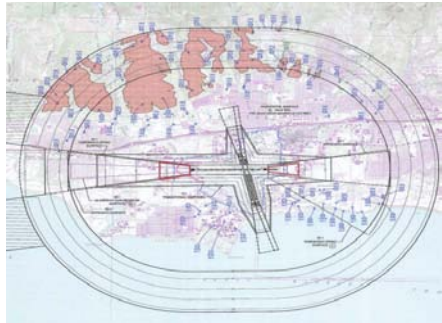


Figure 6. Part 77 Airspace Plan, Santa Barbara Municipal Airport

Given the magnitude and the complexity of the new Daxing International Airport runway layout, development of the Part 77 Airspace Plan would undoubtedly be a complex task. Comprehensive surveys would need to be conducted to ensure the new airport site is clear of existing and potential obstructions and any other aircraft hazards.

OVERFLIGHT

Overflight can be a sensitive issue to many people, especially when occasional noisy business jets and helicopter flights occur. Other people can be annoyed by frequent aircraft overflight, even at low noise levels. At many US airports, complaints often come from people living underneath the aircraft traffic patterns.

ALUCs in the U.S. are limited in their ability to deal with overflight as they have no influence over how an airport operates, nor do their policies affect existing land uses. As part of the airport land use compatibility planning, establishment of an overflight boundary could be an effective airport land use planning option. Figure 7 is an example of a preliminary overflight boundary developed for the Santa Barbara Municipal Airport. Development of such a boundary can be developed by using flight pattern data inputs, airport influence area boundary, noise contours, and modifications of flight patterns and altitudes, and even overflight complaint patterns. Overflight boundary should extend beyond the well-defined NEM contours. Such a policy decision could help notify local communities about the presence of overflight impacts so that residents could make informed decisions regarding acquisition or leasing of properties in the affected areas. In California, state law requires that information regarding the presence of an airport in the vicinity be a part of the disclosure for residential real estate transactions.

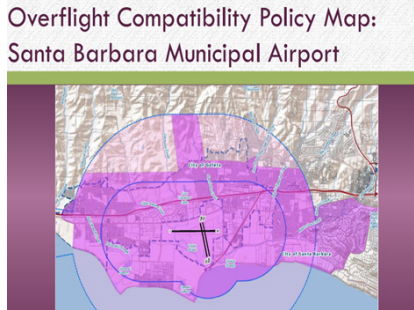


Figure 7. Overflight Compatibility Policy Map, Santa Barbara Municipal Airport

The new Beijing Daxing International Airport is the world's biggest airport ever to be built to-date. Given its enormous land size, the issue of overflight would require comprehensive examination of designated aircraft traffic patterns on existing and future land uses for both civil and military aircraft operation. Continued consultation with local communities for establishment of an airport influence boundary in the Daxing District would be valuable. Real estate notices from home buyer awareness programs would be an effective public relation measure in minimizing potential annoyance concerns. Local airport authorities and provincial governments should consider instituting possible land use policies to develop an overflight boundary as part of the land use compatibility planning strategy to raise the awareness of the new airport. Early and continued consultation with all stakeholders and local community leaders, and seeking technical assistance from consultants would be recommended.

CONCLUSION

China is building 50-60 airports in the next five years. The country would undoubtedly face enormous challenges on issues of airport land use compatibility planning. The tight timeframe of building of the new Beijing Daxing International Airport by 2018 further exacerbates the pressure for implementing a comprehensive airport land use compatibility strategy. The goal in seeking a right balance between promoting the economic vitality of the new airport and protecting and preserving the environment and safety of nearby communities would be an enormously challenging undertaking.

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The Next Generation of Air Transportation System (NextGEN)

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ABSTRACT

The Next Generation Air Transportation System (NextGEN) is the future of the US National Airspace System. Because of its revolutionary innovation of new technologies, NextGEN is still in its infancy on how it would change the aviation industry. This paper presents an overview of NextGEN and its effects on the aviation industry as a whole. The paper compares the current technology of the air transportation system with NextGEN and examines how the latter would affect airports of all sizes, aircraft operation, and the aviation industry. Several key elements are addressed. The “NextGEN for Airports” highlights the awareness and the role of future airports on how it may change the airport infrastructure, industry characteristics and aviation economics. The latest innovation of aircraft technologies are discussed focusing on how these innovations would interact with future NextGEN airports. Finally, the costs and implementation timeline of NextGEN are also addressed.

INTRODUCTION

The air transportation system is vital to our economy. According to Dr. Karlin Toner, Director, Joint Planning and Development Office, the U.S. aviation contributes 5.6 percent of the nation’s GDP. About a third of all US exports are shipped by air and the total number of international air travelers is expected to triple by the year 2025 1/. The reality is that our nation’s aviation system is approaching a critical stage. Airport congestion is approaching to a gridlock. Air travel delays are staggering. The need to move people and goods quickly and efficiently coupled with the arrival of the new generation aircraft such as the B787 Dreamliner, the Airbus A380 Super Jumbo, etc. have added an extra dimension demanding for an ever-changing need to revolutionize our outdated air transportation system.

This paper examines the existing air transportation system and presents the next generation of air transportation system (NextGEN) that would revolutionize our nation’s air transportation. The federal government has been studying various new concepts and technologies of NextGEN for almost a decade now. However, with the amount of new technologies and innovation that need to be tested, NextGEN is still in its experimental stage on how it would change the aviation industry.

THE CURRENT AIR TRANSPORTATION SYSTEM

Currently, our air transportation system is still largely a ground-based navigational system which relies on radar and beacons on the ground. Radio communication is still the primary tool for controllers to communicate with pilots, directing air traffic and guiding them from one ground waypoint to another. Jets are still flying on designated airways under strict horizontal and vertical separations. This current system has been used for decades. It is slow to react to the continued upsurge in air travel demand and often exacerbates the congestion and delays. The fact is that the current system is significantly outdated with WW II technologies. According to the Federal Aviation Administration (FAA), as many as 14 of our major metropolitan area airports will inevitably require additional capacity by 2025; or they will face total gridlock and unsustainable delay costs (see Figure 1).



Figure 1. Current Air Transportation System

The United States has been studying and frantically testing new NextGEN initiatives utilizing state-of-the-art ground- and satellite-based technologies to revolutionize the entire air transportation system. Over the last decade, incremental experiments have been tested using new air traffic control (ATC) concepts. But the process was slow and plagued with delays due to the downturn of the economy, budget crises and the lack of investment. However, the FAA is forging forward with NextGEN initiatives with commitment to switch from the current ground-based system to a satellite-based system to meet our nation's needs.

NEXTGEN FOR AIRPORTS

NextGEN is a vision aimed at transforming the aviation industry. It represents an evolution from a ground-based air traffic control system to a satellite-based air traffic management system, a revolutionized concept which is vital to meet the nation's needs. NextGEN for Airports is a combination of system programs and innovations that heighten the awareness and meeting new challenges of future airports, change aviation economics, and revolutionize the industry characteristics and airport system infrastructure. From a broad perspective, NextGen encompasses the following board initiatives (see Figure 2):



Figure 2. NextGEN Air Transportation System

- **The Optimization of Airspace and Procedures in the Metroplex (OAPM) Program**

As part of the FAA’s fast-track initiative, the FAA is testing the implementation of Performance Based Navigation (PBN) procedures and airspace improvements aiming to provide greater flexibility and more dynamic air traffic management with benefits of reducing fuel consumption and emissions. PBN uses satellite-based navigation and on-board aircraft equipment to navigate with greater precision, providing shorter and more direct flight paths, and improving arrival rates via enhanced controller productivity and safety with minimal adverse environmental effects.

Taking advantage of new equipment installed on aircraft, the idea is a combination of airspace redesign and the development of an area navigation (RNAV). A total of 13 “metroplexes” or geographic areas with multiple airports and complexes, are to be redesigned over the near term, requiring performance-based sensors and GPS procedures to restructure airspace. This year, the first three sites (Houston, North Texas and Washington, D.C.) will complete their designs and enter into the implementation phase. The FAA intends to finish work by the end of Fiscal Year 2016 (see Figure 3).

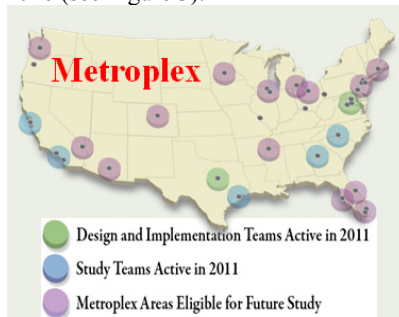


Figure 3. Optimization of Airspace and Procedures in the Metroplex

- **Automatic Dependent Surveillance–Broadcast (ADS-B) Program**

This is the successor to current ground-based radar for tracking aircraft in the National Airspace System (NAS). It makes use of GPS and Wide Area

Augmentation System technology to steadily deploy more than 500 ground stations to provide traffic and weather information for properly equipped aircraft and to support the 28 Terminal Radar Approach Control (TRACON) facilities in the nation.

- **Airport Surface Detection Equipment (ASDE) – MODEL X**

This new technology tracks the movement of aircraft and equipped vehicles on the airport surface. It provides highly accurate surface surveillance tracking aircraft locations, queues and sequencing, and designating paths for efficient use of runways, taxiways, and entry/exit points. ASDE-X will improve airport safety and reduce runway incursions.

- **En-Route Automation Modernization (ERAM)**

This is a new automation platform for controllers to control high-altitude traffic. As a vital part of NextGEN, it involves several systems including the System Wide Information Management, Data Communications, and Automatic Dependent Surveillance-Broadcast (ADS-B). The latter, in particular, depends highly on successful ERAM deployment.

- **Data Communications (Data Comm)**

Data Comm is a critical component in the implementation of NextGen. It is a digital data exchange between air traffic controllers and pilots involving every sector of the air traffic control system. It is the first phase of the transition from the current analog voice system to a digitalized system aiming in order to reduce oral communication and time between the tower and flight crew. Beginning in 2016, part of the Data Comm implementation is the Common Support Service-Weather (CSS-Wx) which uses the National Oceanic Atmospheric Administration (NOAA) radar, sensors, and forecast models to disseminate weather information via the System Wide Information Management (SWIM) to airports, aircraft, and other crucial system links under one common standardized platform.

- **Collaborative Decision Making Process**

The FAA's newly established Surface Operations Office has been conducting work in 2012 among air traffic controllers, flight crews, airports and airline managers and planners. The office validates and refines the operation through a series of simulations of Surface Concept of Operations (ConOps). The ConOps focuses on improved predictions of capacity versus demand at individual airport with more frequent updates ranging from airlines departure schedules to information sharing with all stakeholders to optimize demand-capacity imbalance during over-capacity peaks.

Contracts of six NextGEN transformational programs including ADS-B, System Wide Information System Management (SWIM), CSS-Wx, Collaborative Air Traffic Management Technologies, NAS Voice Switch and Data Comm were already underway.

NEXTGEN AHEAD

In the next few years, NextGEN capabilities and procedures at specific locations will continue to improve its predictability, throughput, and efficiency. New aircraft engines and designs will continue to improve their environmental performance in

terms of noise and emission reduction. The successful implementation of airport surface detection and monitoring technology, the **Airport Surface Detection Equipment Model X**, currently experimenting at New York John F. Kennedy and collaborative Departure Queue Management at Memphis will be expanded to other major airports. Other new technologies and innovations also include the following:

- **Terminal Flight Data Manager (TFDM):** Capabilities are planned for 2017 to transition to electronic flight data exchange between tower and terminal Radar Approach Control (TRACON) in order to enable more efficient traffic flow movements.
- **Re-Categorizing Wake Turbulence and Standards:** This new procedures will reduce the necessary arrival spacing among various classes of aircraft, thus increasing airspace capacity. Equally vital to flight planning is the real-time information on special uses of airspace and runways using digitized “Notices to Airmen” (NOTAM).
- **FAA Oceanic Automation System (Ocean21):** This system will enable aircraft to take fuel saving altitudes. When aircraft are equipped with Automatic Dependent Surveillance-Contract (ADS-C), the GPS signals will be able to verify aircraft position and provide climb/descend procedure (CDP). Dependent Surveillance-Broadcast (ADS-B) In-Trail Procedures (ITP) derived from the flight deck will relay information to the controller, enabling aircraft with desired climb and descend altitudes.
- **NextGEN Weather Detection and Forecast:** These capacities will improve air traffic planning and operation. The in-flight icing, turbulence, ceiling and visibility, and a gridded national ceiling and visibility forecasts are expected by 2016. The NextGEN Weather Processor (NWP) will establish a common weather processing platform for a full operation by 2016.

NEW AIRCRAFT TECHNOLOGY HIGHLIGHTS

The second set of NextGEN key components focuses on introducing the latest innovations of aircraft technology and how they can closely interact with future airports. New navigational equipment and avionics onboard the new generation aircraft will be a big part of NextGEN implementation. Advanced on-board aircraft navigational equipment and avionics will transmit, receive, and interpret information to enable transformation from a ground-based to a satellite-based system (Figure 4).



Figure 4. New Aircraft Technology

Flight Planning will access relevant information through a common network source to handle airport congestion and airspace constraints. Operators will plan their flights with a full picture of ground and air routes as well as flight trajectory. These advances will mean efficient air traffic management and enhanced environmental performance for flights.

Data communications and pre-departure clearances will allow amendments to flight plans for push back, taxi and departure. Flight deck displays would indicate the aircraft position. Controllers can visualize the surface aircraft movements and assign the best available ground paths to enhance safety and efficiency, particularly under conditions of low visibility. Radio transmissions and operational delays will keep delays to a minimum (see Figure 5).

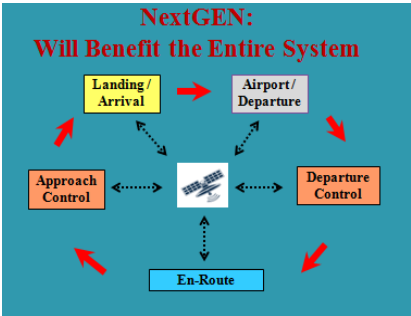


Figure 5. NextGEN will Benefit the Entire System

With reduced separation standards en-route, flight crew will be able to take up some spacing assignment responsibility, from monitoring of their aircraft position during all phases of their flight to allowing Data Comm to provide strategic en-route information. Controllers will be able to use multiple precision paths to enhance the flow efficiency to each runway at the destination airport. The use of RNAV and the Required Navigation Performance (RNP), the onboard performance monitoring systems will provide alerting capability on board the aircraft.

Before the flight lands, specific runway, taxiways and path will be available to flight crew via Data Comm. Flight deck and controller displays will monitor aircraft movements. Airport and ramp operators will have accurate information about the inbound projected aircraft arrival time at the gate.

The *ADS-B Out* avionics will receive aircraft positioning data from GPS satellites, process them and transmit them to the ground. Ground stations will send data to controller's displays on the ground and to cockpit displays that are equipped with *ADS-B In*.

RNP will allow aircraft to fly tightly defined corridor of airspace. A combination of Area Navigation (RNAV) and RNP will provide increased airport throughput capacity and enhance safety. New RNP procedures are being designed for tests at Anchorage, Buffalo, N.Y., Dayton, and Milwaukee airports for completion by this year (2014).

The FAA has contracted with United Airlines and JetBlue Airways to demonstrate ADS-B benefits. United is operating In-Trail Procedures (ITP) on routes over the Pacific Ocean using aircraft equipped with ADS-B Out and ADS-B In. Flight crews of these aircraft are aware of the location, speed and identity of other aircraft in the demonstration program and know when to request from controllers to climb to a more fuel-efficient altitude. JetBlue's demonstration involves operating ADS-B-equipped aircraft on N-S routes off the United States' East Coast.

THE COST OF NEXTGEN

NextGEN will no doubt transform the National Airspace System. Deployment of NextGEN will involve cooperation from the entire aviation community. Airline operators will have to equip their aircraft, develop procedures, and train flight crew and maintenance personnel to take advantage of NextGen capabilities and benefits.

While the cost of NextGen is enormous, the FAA is considering operational and financial incentives for aircraft equipage. In 2011, part of the NextGEN implementation is to provide opportunities for NextGen-capable aircraft to receive better services and derive benefits directly from operations that use these capabilities. The FAA is aiming for a "critical mass" for NextGEN-capable aircraft for benefits to be attained.

In 2012, the FAA is pursuing a concept of: Aircraft Priority Access Selection Sequence (AirPASS). Aircraft and airline operators with NextGen capabilities would receive priorities for operations such as takeoffs, approaches, reroutes and releases from temporary flight restrictions.

The FAA Reauthorization Act encourages public-private partnership, leveraging and maximizing the use of private-sector capital for financing equipage of aircraft. The FAA Modernization and Reform Act of 2012 allows for the establishment of an avionics equipage financial incentive program. The FAA also provides guarantees for private-sector loans to all aviation partners. NextGEN economic value depends on the rate of advancing capabilities on both government and industry deployment across the National Airspace System.

NEXTGEN TIMELINE

According to the NextGEN implementation schedule, 2018 to 2020 mark the end of the mid-term. NextGEN is an advanced integrated system. Many of its component systems are mutually dependent on one or more other systems. For example, the delivery ADS-B depends on the success implementation of the ERAM functions. ERAM is instrumental to the on-time implementation of Data Comm and SWIM. The budget crises and various challenges from various technical tests have caused delays in the original implementation schedule. The long-term implementation timeline is unknown at this point, which in turn further hinders the progress and other capabilities of the system integration. Some speculate that NextGEN may not be fully realized until 2025.

CONCLUSION

NextGEN is crucial to the future of the National Airspace System. It affects airports of all sizes, aircraft operation, and the entire aviation industry. The U.S. is determined to strive forward with a flexible plan toward a 2025 vision and beyond. The ability for pilots to efficiently handle more information together with greater air traffic control automation would help optimize safety and efficiency. Advanced on-board computers would lead to better pilot decisions on responsibilities, aircraft operating performance and piloting aircraft.

The FAA is fully committed in transforming the nation's aviation system, achieving a higher level of safety, and setting strategic priorities for modernizing our air transportation system. NextGEN has started almost a decade ago. Full implementation in the next 10, 20 years will be crucial to get us there.

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Railway Materials Distribution Business Operation Model Selection Research

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ABSTRACT

With the development of high-speed railway construction, existing services cost generally high, delivery time long, poor services quality, affecting the lifting Materials distribution service level of China Railway. In this paper, in order to improve the scientific decision making in China Railway Materials Distribution System, making reference to relevant research field of the latest achievements, research on the Materials Distribution related decision problems. In this paper, the comparative analysis of the railway logistics operation mode is to be done, put forward index system of "mode selection better", and then with the traditional AHP and TOPSIS method, operation mode optimization model is improved and constructed based on GAHP and GTOPSIS. Research shows that, the system of railway Materials Distribution, the selection of service operation mode is practical, effective, can help enterprises to choose the best operation mode that match their needs.

Keywords: railway, materials distribution, AHP, TOPSIS

1 INTRODUCTION

1.1 The research background and significance

With the development of high-speed railway construction, China has integrated into the trend of the times of world high-speed railway. In 2013, the arrangement of 6500 billion for investment in fixed assets, including infrastructure investment for 5200 billion. From January to April in 2013, the investment in railway construction growth 25% year-on-year, the basic construction investment of 545.1 billion is finished in the first quarter of the year (The Chinese railway website, (2013). The railway material distribution has been developed for a very long time in China, but with the business philosophy development and the application of advanced technology, in the process of railway construction will still face many unprecedented problems. The purpose of this study is to analyze and solve the important decision problem, research on the operation model evaluation index system of railway material distribution and selection mode can provide scientific basis for enterprises to develop logistics strategy, causes the enterprise to find

material distribution operation mode is the most suitable for their own, and enhance their core competitiveness.

1.2 The research status at home and abroad

Operation mode of material distribution service has been the focus of many scholars research and attention. In 2003, an article in the "comparative study of the logistics distribution mode in the developed countries and regions", Wang Qing presented some reference experience of process to build logistics distribution system in our country through the analysis (Wang, 2003). In 2004, an article in the "research on the mode of network logistics distribution", Xu et al. (2004) proposed cooperative commodity distribution mode, for example, distribution center oriented by retail business, distribution center by storage and transportation as the dominant, distribution center dominant by wholesale, distribution center leading to the convenience store service and other forms. A University of Tennessee study found that, the third party logistics distribution mode can make the logistics cost of enterprises reduce by 62%, core business concentration of the enterprise increased 56%, raising the service level of 62%, assets reduced 48%, employee expenses reduced 50% (Cai, 2011). Quinn et al. (1994) proposed that enterprises should evaluate whether the various business activities have become or will become the best in the world, for those who cannot become the world's best business module, should resolutely cancel, outsourcing or cooperate with others. Ballow (1999) proposed a two-dimensional decision way, in his view, the enterprise logistics business operation mode should consider from two-dimensional, logistics management level of the enterprise and logistics influence to the enterprise.

2 SUMMARY OF RAILWAY MATERIALS DISTRIBUTION BUSINESS

Material distribution is an important link in the operation of railway related enterprise business. This chapter will analysis the current situation and main problems of railway materials distribution business, and put forward the solutions to the problems.

2.1 Current situation of railway materials distribution business

From the perspective of the actual business operation, this article will break the progress of railway materials distribution business into information platform processing, distribution center disposing, transporting and reaching to the users. The four steps can be seen in Figure 1 as following.

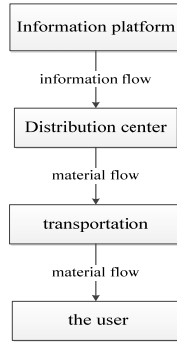


Figure 1. Flow Chart of Railway Materials Distribution Business

The characteristics of railway materials distribution business can be summarized as follows:

- (1) Strong timeliness.
- (2) Larger Numbers of Orders.
- (3) Wide user regional distribution.
- (4) High standard for workers' quality.

2.2 Problems analysis of railway materials distribution business

- (1) High cost on service.
- (2) Long material delivery period.
- (3) Poor services.

2.3 Solutions to the problem of railway materials distribution business

Focusing on the problems in the railway materials distribution business, this essay will offer basic solutions from the perspectives of railway materials distribution business operation model.

The material distribution business operation mode in this paper refers to the operation ways from the view of self-shipment and outsourcing. As for the problem of high cost and long service period, it is mainly because of most products form material manufacturers are single and simple and cannot meet the requirement of large and wide distribution network. Therefore, related railway materials enterprises should choose suitable business operation mode including wholly-owned-shipment, wholly-outsourcing, or the combination of own-shipment and outsourcing. Through scientific operation mode, it can fundamentally reduce the distribution cost, shorten the goods delivery time and reduce the gap between actual delivery time and consumer expectations.

3 SELECTION OF THE OPERATION MODES OF RAILWAY MATERIALS DISTRIBUTION SERVICES

3.1 Selection principles of the operation mode of railway materials distribution services

Through the collaboration and coordination of various links including warehousing, transport and delivery, the distribution system of railway materials aims to ensure the corporate sale of goods and provides customers with the material distribution services as the corporation expects at a reasonable cost and a relatively high efficiency. On that basis, this research concludes the selection principles of the operation mode of distribution services of railway materials into the following aspects:

- (1) Conform to the development strategies.
- (2) Improve the effectiveness of material distribution system.
- (3) Keep operational risks under control.

3.2 Comparative analysis of the operation mode of railway materials distribution services

According to different criteria, there are various sorting techniques in terms of the operation mode of railway materials distribution services. Referring to the previous research achievements, this research combines the features of practical business operation in relevant railway corporations, whereby dividing the operation mode into the following six types from the perspective of the constitution of the main operational body of material distribution services:

- (1) Total self-operation mode.
- (2) Internal outsourcing mode.
- (3) Logistics alliance mode.
- (4) Partial outsourcing mode.
- (5) “Take-over by system” mode.
- (6) Total outsourcing mode.

Briefly compared the six modes mentioned above, we work out Table 1:

Table 1. Comparison of the Operation Modes of Railway Material Distribution Services.

Compared factors	Total self-operation mode	Internal outsourcing mode	Logistics alliance mode	Partial outsourcing mode	“Take-over by system” mode	Total outsourcing mode
Transaction costs	low	relatively low	medium	medium	high	high
Service reliability	high	high	medium	medium	low	low
Service capability	low	low	medium	medium	high	high

Actually, the six operation modes mentioned above both possess their own advantages, neither superior nor inferior to each other. Each of them suits different conditions, and hence relevant railway material corporations should choose the operation mode according with their own development strategies on the basis of their practical situation.

3.3 Design of the evaluation index system of the railway materials distribution business model

3.3.1 Constitution of the evaluation index system

As shown in Figure 2, this paper focusing on formulating the evaluation index system of the railway materials distribution business model (Cai, 2011). The formulating process begins with the main task undertaken by the railway materials distribution system, combined with the choosing principle of the materials distribution business model and the characteristics of each model.

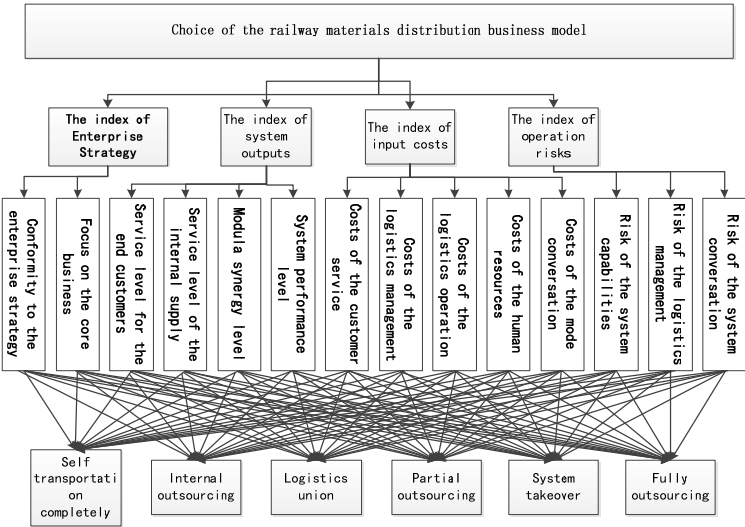


Figure 2. Evaluation Index System of the Railway Materials Distribution Business Model

3.3.2 Scoring rules of the indexes

According to the index system stated before, this research concludes the scoring rules of the indexes as shown in Table 2.

Table 2. Scoring Rules of the Evaluation Indexes of the Railway Materials Distribution Business Model

First-level indexes	Second-level indexes	Scoring rules
Enterprise strategy	Conformity to the enterprise strategy	The full mark is 4, and the lowest is 1. 4- very consistent; 3-well consistent; 2-common consistent; 1-not consistent.
	Focus on the core business	
System outputs	Service level for the end customers	The full mark is 5, and the lowest is 1. 5-top level; 4-high level; 3-middel level; 2-low level; 1-poor level.
	Service level of the internal supply	
	Modula synergy level	The full mark is 4, and the worst is 1. 4-top level; 3-high level; 2-ordinary level; 1-not consistent.
	System performance level	
Input costs	Costs of the customer service	The full mark is 4, and the worst is 1. 4-reduce the costs significantly; 3-reduce the costs; 2-unchanged; 1-raise the costs.
	Costs of the logistics management	
	Costs of the logistics operation	
	Costs of the human resources	
	Costs of the mode conversation	The full mark is 4, and the worst is 1. 4-the conversation costs could be ignored; 3-common conversation costs; 2-high conversation costs; 1-very high conversation costs.
Operation risks	Risk of the system capabilities	The full mark is 4, and the worst is 1. 4-no risks; 3-low risk; 2-general risk; 1-high risk.
	Risk of the logistics management	
	Risk of the system conversation	

3.4 Selection model of the railway materials distribution business model based on GAHP-GTOPSIS

3.4.1 Index weight determination of the materials distribution business model based on GAHP

Choose and determine the decision expert sets and their weights.

Determine the decision expert sets:

$$T = \{T_1, T_2, \dots, T_N\} \quad (3-1)$$

Determine the weights of the experts using Five-Scale-Method.

Suppose the Five-Scale-Value of the Expert T_i , with respect to the Expert

T_j , is t_{ij} . ($t_{ij} = 4 \sim 8$), thus the authority judge matrix is:

$$T = (t_{ij})_{N \times N} \quad (3-2)$$

The weight of the Expert T_i is:

$$\mu_i = \sum_{j=1}^n t_{ij} \quad i = 1, 2, \dots, N \quad (3-3)$$

$$w_i = \mu_i \sum_{j=1}^n \mu_j \quad i = 1, 2, \dots, N \quad (3-4)$$

$$w^{(T)} = (w_1^{(T)}, w_2^{(T)}, \dots, w_N^{(T)})^T \quad (3-5)$$

The individual judge matrix of the Expert j is:

$$A^{(j)} = (a_{mi}^{(j)})_{n \times n} \quad (3-6)$$

in which:

$$a_{mi}^{(j)} > 0, a_{im}^{(j)} = \frac{1}{a_{mi}^{(j)}}, a_{mm}^{(j)} = 1 \quad (3-7)$$

Consistency test:

The maximum eigenvector $\tau_{\max}^{(j)}$ of $A^{(j)}$ satisfies:

$$A^{(j)} w^{(j)} = \tau_{\max}^{(j)} w^{(j)} \quad (3-8)$$

The index of consistency:

$$C.I. = \frac{\tau_{\max}^{(j)} - n}{n-1} \quad (3-9)$$

The consistency ratio

$$C.R. = \frac{C.I.}{R.I.} < 0.1 \quad (R.I. \text{ is the average of the random consistency indexes}) \quad (3-10)$$

Using weighted geometric mean method,

$$a_{mi} = \prod_{j=1}^n (a_{mi}^{(j)})^{w^{(j)}} \quad (3-11)$$

$$A = (a_{mi})_{n \times n} \quad (3-12)$$

$Aw = \tau_{\max} w$ ($\tau_{\max}$ is the maximum eigenvalue of A , w is the corresponding eigenvector) (3-13)

Let $P_j = (P_{j,1}, P_{j,2}, \dots, P_{j,n_{j-1}})^T$, a $n_j * n_{j-1}$ matrix, which represents the weight vector of the j -line indexes with respect to the $j-1$ line indexes.

$$w_j = (w_{j1}, w_{j2}, \dots, w_{jn_j})^T = P_j w_{j-1} \quad (3-14)$$

$$w_{jm} = \sum_{i=1}^{n_{j-1}} P_{j,m,i} w_{j-1,i}, m=1, 2, \dots, n \quad (3-15)$$

$$C.I._j = (C.I._{j,1}, C.I._{j,2}, \dots, C.I._{j,n_{j-1}}) w_{j-1} \quad (3-16)$$

$$R.I._j = (R.I._{j,1}, R.I._{j,2}, \dots, R.I._{j,n_{j-1}}) w_{j-1} \quad (3-17)$$

$$\text{When } C.I._j = \frac{C.I._j}{R.I._j} < 0.1, \text{ the requirement is satisfied.} \quad (3-18)$$

By computing out the comprehensive evaluation value of each business mode, sequence them in descending order, and then the business mode, which has the maximum value, will be the most matched materials distribution business mode with the enterprise reality.

3.4.2 Based on GTOPSIS railway materials distribution business operation mode optimization method

Decision-making expert set $T = \{T_1, T_2, \dots, T_N\}$. structure experts individual decision matrix:

$$S^{(j)} = (s_{mi}^{(j)}), \text{ in } j=1, 2, \dots, N; m=1, 2, \dots, k; i=1, 2, \dots, n$$

$$D_{mi} \left(s_{mi}^{(j)}, s_{mi}^{(l)} \right) = \frac{\min(s_{mi}^{(j)}, s_{mi}^{(l)})}{\max(s_{mi}^{(j)}, s_{mi}^{(l)})} \quad (3-19)$$

The greater the value of $D_{mi} \left(s_{mi}^{(j)}, s_{mi}^{(l)} \right)$ show experts j and l have a greater the similarity for the m model evaluation in the first i indicators.

$$I_{mi}^{(j)} = \frac{1}{N} \sum_{l=1}^N D_{mi} \left(s_{mi}^{(j)}, s_{mi}^{(l)} \right) \quad (3-20)$$

$$S = (s_{mi})_{k \times n} \quad (3-21)$$

$$s_{mi} = \frac{\sum_{l=1}^N (I_{mi}^{(j)} + s_{mi}^{(j)})}{\sum_{l=1}^N I_{mi}^{(j)}} \quad (3-22)$$

$$\text{Forward indicator } s_{mi}^d = \frac{s_{mi}}{\max_m s_{mi}} \quad (3-23)$$

$$\text{Contrary indicator } s_{mi}^d = \frac{\min_m s_{mi}}{s_{mi}} \quad (3-24)$$

$$\text{Standardized expert group decision matrix } S^w = (s_{mi}^d)_{k \times n} \quad (3-25)$$

$$S^w = (s_{mi}^w)_{k \times n} = S^w \quad (3-26)$$

$$\text{The ideal solution: } A^+ = (s_i^+) = \{(\max_{1 \leq m \leq k} s_{mi}^w | i \in J) | m = 1, 2, \dots, k\} \quad (3-27)$$

The negative ideal solution:

$$A^- = (s_i^-) = \{(\min_{1 \leq m \leq k} s_{mi}^w | i \in J) | m = 1, 2, \dots, k\} \quad (3-28)$$

$$\text{Model } m \text{ and the ideal solution distance: } D_m^+ = \sqrt{\sum_{i=1}^n (s_{mi}^w - s_i^+)^2} \quad (3-29)$$

Model m and the negative ideal solution distance:

$$D_m^- = \sqrt{\sum_{i=1}^n (s_{mi}^w - s_i^-)^2} \quad (3-30)$$

Close to the degree of each mode and the ideal solution A^+ :

$$C_m^+ = \frac{D_m^-}{D_m^+ + D_m^-} (m=1, 2, \dots, k) \quad (3-31)$$

According to the each model and the positive ideal solution close to the degree ordering. Close to the degree of numerical minimum model is the best railway materials distribution business operation model.

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A Study of the Travel Mode Choice Model of Chinese Urban Elderly

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ABSTRACT

This paper is based on resident travel survey data in Jinan city using a disaggregate model and conducts exploratory research related to the travel mode choice behavior of the elderly. The results show that age, gender, having a driver's license and a monthly pass, number of bikes and electric cars in the family, travel purpose, travel distance, travel time, and other variables have an important impact on the travel mode choice of the elderly. After determining the characteristics of the variables, the parameters of the model were calibrated. According to the model calibration results, combined with the sample data used in the parameter calibration, the future travel choice probability of the elderly was predicted. The result of personal choice was calculated using aggregate analysis after predicting the average probability of all types of travel modes and comparing them with the actual survey sample selection ratio. The final model errors were below 2 percent, which is within the allowable range, so the model parameter calibration is effective.

INTRODUCTION

Since the 1980s, the long-term implementation of China's one-child policy has resulted in a low birth rate and rapid economic growth, as well as scientific and technological progress and improvement of medical and health conditions and living standards. The average life span has improved constantly; therefore, the age structure of China's population has started to incline toward aging.

Currently, there are more than 177 million older people (defined as people age 60 or older) living in China, comprising 13.26 percent of the Chinese population, and including 110 million people age 65 or older, comprising 8.87 percent of the Chinese population. China has the highest elderly population in the world. Experts predict that by the year 2050, the population of people age 60+ is expected to be more than 400 million, comprising 25.5 percent of the Chinese population. This rapid growth of the older population will create an environment where one out of four people in China will be age 60 or older, and China will become the highest-aging country. Jinan city, as the capital city of Shandong Province, of which the population age 65 or older has reached 0.62 million, accounts for 9.15 percent of the resident population and higher than the national proportion of the population over age 65, is a seriously aging city.

Population aging will have a major impact on all aspects of social life, and one of the most interesting is the ever-growing number of elderly participants in traffic. As an important and indispensable component of transportation, their traffic behavior will have a major impact on the entire traffic condition. To study elderly travel behavior through choice of travel mode can accurately provide an understanding of urban elderly travel characteristics and provide a basis for planning and solving traffic problems in an aging society.

This paper consists of three main sections. First, according to a resident travel survey data in Jinan city, it establishes the elderly travel choice model using the disaggregate method. Then, it sets the model calibration using the disaggregate data, and the model calibration result is verified. Finally, a summary of key conclusions is provided.

CASE ANALYSIS

Elderly travel mode choice of ML model. According to a 2009 resident travel survey in Jinan city, elderly travel modes include walk, bike, electric car, bus, taxi and private car. According to theory related to the disaggregate model, this study selects these six modes of transportation as a choice and sets up an elderly travel mode choice of an ML model. The form of model is as follows:

$$P_{in} = \frac{e^{V_{in}}}{e^{V_{in}} + e^{V_{2n}} + e^{V_{3n}} + e^{V_{4n}} + e^{V_{5n}} + e^{V_{6n}}}$$

Where $V_{in} = \sum_{j=1}^k \theta_{inj} X_{inj} + \theta_i$ is the fixed part of the utility function that elderly n choose way i to travel, and θ_i is independent of the fixed dummy with n .

Parameter selection. In this study, from domestic and foreign research results, while analyzing the influencing factors of travel mode choice, the main considerations are individual properties and the differences in travel properties. Individual properties are divided into personal attributes and family attributes (personal socio-economic factors and family's socio-economic factors).

Before model calibration, especially when independent variables of factors are numerous, the independent variables must be screened because not all the factors influence the outcome; if a variable has no effect on the dependent variable results, but also substitutes in the equation with other variables, it may affect the other variables in the model of the correct calibration, so the initial screening of the independent variable is particularly important. Table 1 shows a contingency analysis indicating the correlation between discrete variables and mode choice. As can be seen in the table, aside from the variable of Number of Cars, which has a concomitant probability of 0.099, most have a 0.01 significance level, so the number-of-cars variable has no impact on travel mode choice and is excluded. The rest of the attributes have a significant probability of less than 0.01, so, in addition to the characteristics of the car-number variable, the other characteristics all have a certain influence on travel mode choice. ML model variable settings are shown in Table 2.

Table 1. Different Attributes -Travel Mode Analysis Cross-Analysis.

Variables		Travel Mode						Total	Chi-square value	Deg. of freedom	Concomitant prob.
		Walk	Bike	Elec. car	Bus	Taxi	Car				
Age	60-65 yrs	797	124	83	306	4	18	1332	106.113	25	0.000
	66-70 yrs	363	37	27	88	4	14	533			
	71-75 yrs	198	14	5	87	4	0	308			
	76-80 yrs	148	2	1	48	0	0	199			
Gender	>81 yrs	46	2	0	2	0	0	50	73.727	5	0.000
	Male	732	123	91	288	5	23	1262			
	Female	820	56	25	243	7	9	1160			
Educ. level	Primary and below	462	37	18	105	2	8	632	89.138	20	0.000
	Junior high	482	56	31	187	7	4	767			
	High	463	57	59	147	3	14	743			
	University	139	29	8	92	0	6	274			
	Graduate +	6	0	0	0	0	0	6			
Driver's license	Yes	56	21	1	19	0	15	112	159.277	5	0.000
	No	1496	158	115	512	12	17	2310			
Monthly ticket	Yes	1387	160	90	389	12	22	2060	97.379	5	0.000
	No	165	19	26	142	0	10	362			
Family monthly income	<1000 Yuan	70	1	6	10	0	0	87	179.833	55	0.000
	1000~1999Yuan	206	14	11	50	8	0	289			
	2000~2999Yuan	329	47	25	117	0	6	524			
	3000~3999Yuan	297	40	25	131	1	0	494			
	4000~4999Yuan	176	21	14	69	0	6	286			
	5000~5999Yuan	127	13	15	35	0	4	194			
	6000~6999Yuan	99	11	3	48	0	4	165			
# Bikes	>7000Yuan	248	32	17	71	3	12	383	171.513	20	0.000
	0	664	11	46	251	5	24	1001			
	1	612	96	48	214	7	8	985			
	2	216	66	22	54	0	0	358			
	3	60	6	0	10	0	0	76			
# Elec. cars	4	0	0	0	2	0	0	2	259.318	15	0.000
	0	1131	118	8	373	12	22	1664			
	1	367	47	85	125	0	10	634			
	2	44	14	23	33	0	0	114			
# Cars	3	10	0	0	0	0	0	10	16.02	10	0.099
	0	1468	164	103	498	12	32	2277			
	1	78	15	13	29	0	0	135			
Travel purpose	2	6	0	0	4	0	0	10	419.457	45	0.000
	Work	20	7	7	26	0	5	65			
	Return home	574	58	40	212	4	12	900			
	Return trip	176	24	12	50	0	4	266			
	Shopping and entertainment	394	39	20	119	0	8	580			
	See doctor	11	3	0	14	6	3	37			
	Visit friends, relatives	24	4	2	21	0	0	51			
Travel distance	Pick up children	42	21	25	19	0	0	107	1242.947	15	0.000
	Other	311	23	10	70	2	0	416			
	<1 km	1155	35	18	18	0	6	1232			
	1-5 km	359	104	62	247	8	10	790			
	5-10 km	36	33	28	187	0	8	292			
	<10 km	2	7	8	79	4	8	108			

Table 2. Model Variable Settings.

	Factors	Variable	Description
Personal attributes	Sex	Sex	Female: 1, Male: 0
	Age	Age	Divided into three age groups
	Education	Edu	University and above: 1, High school and below: 0
	Monthly ticket	MT	Have: 1, No: 0
	Driver's license	DL	Have: 1, No: 0
Family attributes	# Bikes	BIC	0, 1, 2, ...
	# Electric cars	EBIC	0, 1, 2, ...
	Family monthly income	HINC	Divided into three groups
Travel attributes	Travel distance	TD	Divided into three groups
	Travel purpose	TP	Divided into four groups
	Travel time	TT	Departure and arrival time difference
	Travel cost	TC	Travel expenses during transportation

Model calibration. Based on the above analysis, the final variables and their sub-selection, which will generate the elderly travel choice model, are gained, as shown in Table 3. Model calibration results are shown in Table 4.

Table 3. Model Variable Settings.

		Walk V1n	Bike V2n	Elec. car V3n	Bus V4n	Taxi V5n	Car V6n	Coeff. θ
Alternative specific dummy variables	Xin1	1	0	0	0	0	0	θ1
	Xin2	0	1	0	0	0	0	θ2
	Xin3	0	0	1	0	0	0	θ3
	Xin4	0	0	0	1	0	0	θ4
	Xin5	0	0	0	0	1	0	θ5
Personal attributes variables	Sex	0	0	0	1	0	0	θ6
	Age1	0	0	0	1	0	0	θ7
	Age2	1	0	0	0	0	0	θ8
	Edu	0	0	0	0	0	1	θ9
	MT	0	0	0	1	0	0	θ10
Family attributes variables	DL	0	0	0	0	0	1	θ11
	BIC	0	1	0	0	0	0	θ12
	EBIC	0	0	1	0	0	0	θ13
	HINC1	0	1	0	0	0	0	θ14
	HINC2	0	0	0	0	1	0	θ15
Travel attributes variables	TD1	1	0	0	0	0	0	θ16
	TD2	0	0	0	1	0	0	θ17
	TP1	1	0	0	0	0	0	θ18
	TP2	0	0	0	1	0	0	θ19
	TP3	0	0	0	0	1	0	θ20
Common variables	TT	X1n21	X2n21	X3n21	X4n21	X5n21	X6n21	θ21
	TC	X1n22	X2n22	X3n22	X4n22	X5n22	X6n22	θ22

Table 4. Model Calibration Results.

Variables		Model 1			Model 2		
		Estimate (θ)	Standard error	t value	Estimate (θ)	Standard error	t value
Alternative specific dummy variables	Asc1	10.933	1.004	10.893	5.412	0.48	11.271
	Asc2	1.911	0.953	2.006	-2.208	0.336	-6.576
	Asc3	0.389	1.079	1.961	-3.314	0.592	-5.6
	Asc4	1.49	0.974	2.528	-3.399	0.528	-6.453
	Asc5	3.214	1.008	3.19	-1.61	0.324	-4.968
Discrete variables	Sex	0.35	0.138	2.544	0.299	0.132	2.265
	Age1	-0.931	0.26	-3.577	1.299	0.228	5.698
	Age2	-1.053	0.144	-7.3	0.963	0.224	4.297
	Edu	-0.373	0.511	-0.73	—	—	—
	MT	0.744	0.175	4.255	0.999	0.171	5.843
	DL	-3.271	0.429	-7.623	-2.933	0.379	-7.729
	BIC	1.129	0.102	11.508	0.948	0.098	9.653
	EBIC	1.616	0.141	11.478	1.643	0.139	11.785
	HINC1	0.185	0.13	1.422	—	—	—
	HINC2	1.571	0.371	1.238	—	—	—
	TD1	-1.889	0.138	-13.686	1.643	0.131	-10.179
	TD2	0.986	0.122	8.06	0.943	0.123	7.665
	TP1	-0.555	0.085	-6.559	-0.62	0.088	-7.053
	TP2	0.274	0.297	2.191	-0.108	0.078	-2.365
	TP3	-0.161	0.091	-1.985	-0.004	0.285	-1.983
Continuous variables	TT	-0.036	0.01	-3.489	0.118	0.009	13.565
	TC	113.329	163.277	0.694	—	—	—
LL(0)		-4339.641			-4339.641		
LL(θ)		-1665.654			1539.398		
-2(LL(0)- LL(θ))		5347.975			5600.487		
ρ^2		0.616			0.645		
$\bar{\rho}^2$		0.612			0.641		

After the establishment of the model, it is evaluated whether the model fits the data well, usually with goodness of fit indicators to test. This paper adopts ρ^2 and $\bar{\rho}^2$ to verify; ρ^2 is the goodness ratio; $\bar{\rho}^2$ is the result of goodness ratio after the freedom adjustment. In practice, when ρ^2 and $\bar{\rho}^2$ reach 0.2 ~ 0.4, it indicates that the precision of the model is higher. In addition, when $|t_k|$ is greater than 1.96, there is 95% certainty that the corresponding variable is the selection of factors that affect probability. Conversely, when $|t_k|$ is less than or equal to 1.96 in the 95% reliability level, the corresponding variable does not affect the probability of selection. At this time, variables can be excluded from influencing factors and then parameters are re-estimated.

From the calibration results of the model 1, we can see that education, family monthly income, and travel costs are the three variable t-test values that are less than the absolute value of 1.96 and are not significant, so these three variables were removed and the model calibration was re-opened, namely model 2. In Model 2, ρ^2 is 0.645,

and $\bar{\rho}^2$ is 0.641, so the model is of high accuracy.

Model analysis results. Through the analysis of choosing the model of resident travel, we can know the important influencing factors of mode choice for travel, which is helpful for determining the causes of the transportation mode structure, and we can put forward effective measures to effect and guide the transportation mode structure. This analysis, based on the ML model according to the 2009 residents travel survey data in Jinan, has the following results:

1) People age 75+ are more likely to travel on foot, which may be related to the fact that other methods of travel are more difficult due to increasing age and deteriorating health, so the probability that the elderly choose to travel on foot will be larger with increasing age. When the travel distance is within 1 km, travelers also tend to choose to walk, but with distance increasing, the probability of walk decreases for the elderly. In addition, the travelers for the purpose of shopping or seeking entertainment tend to travel on foot, which is consistent with the initial conclusion that older people tend to shop in closer shopping areas.

2) Whether or not a family has a bicycle is an important reason that affects the elderly choosing that mode, as is whether a family has an electric car.

3) Gender, age, having a monthly pass, travel distance, and the purpose of travel are important factors to affect whether the elderly choose public transport. Those age 60–70 years tend to choose public transport travel, and travelers of distances in the 1–5 km range also tend to choose public transport travel. With increasing distance, the probability of the elderly choosing public transport travel will increase. In addition, for the purpose of visiting a doctor, the elderly are more inclined to choose public transport.

4) Whether or not the travelers have a driving license is an important reason affecting whether or not the elderly choose car travel. However, this has little effect on the elderly choosing the travel mode of car, mainly because the elderly after retirement, faced with the pressure of fewer economic sources, have a low probability of car ownership. Also, even if the family has a car, because of increasing age and body deterioration, they cannot drive, so whether they have driving capability has an important effect on travel choice.

5) From the model calibration results, trip characteristics factors—travel purpose, travel distance, and travel time—have great influence, but travel expenses have no effect on travel mode choice, which is inconsistent with the initial research conclusion that, after retirement, the elderly have plenty of time but little money, so they are concerned more with travel expenses than travel time. This could be because the economic conditions in Jinan city are relatively good, so economic pressure on older people is not significant.

Model aggregate analysis. To set the model for aggregate analysis, we compared the calculated probability value by model with the actual probability to disaggregate the validity of the model by further. According to the model calibration results, combined with the sample data used in the calibration parameters, predicting the future travel choice probability of the elderly, the result of personal choice is calculated to conduct an aggregate analysis. After predicting the average probability of all types of travel modes, we compared the actual survey sample selection ratio, as shown in Table 5.

Table 5 shows that the model error values are below 2 percent, the error within the allowable range, so the model parameter calibration effective.

Table 5. Model Predictions and Actual Observed Value Comparison Table.

Travel mode		Walk	Bike	Electric car	Bus	Taxi	Car
Practical survey (est.)	number	1552	179	116	531	12	32
	ratio	64.08	7.39	4.79	21.92	0.5	1.32
Model calculation (obs.)	number	1549	176	118	536	12	31
	ratio	63.94	7.27	4.88	22.12	0.49	1.3
Error (est.-obs./obs.)		0.22%	1.66%	-1.86%	-0.89%	1.11%	1.63%

CONCLUSION

This paper is based on a resident travel survey data in Jinan city using the disaggregate model; exploratory research was conducted for the travel mode choice behavior of the elderly. The results show that age, gender, having a driver’s license and a monthly pass, number of bikes and electric cars in the family, travel purpose, travel distance, travel time, and other variables have an important impact on the travel mode choice of the elderly. After determining the characteristics of variables, the parameters of the model were calibrated. According to the model calibration results, combined with the sample data used in the parameter calibration, the future travel choice probability of the elderly was predicted. The result of personal choice was calculated to conduct an aggregate analysis; after predicting the average probability of all types of travel modes and comparing it with the actual survey sample selection ratio, the final model error values are below 2 percent, which is within the allowable range, so the model parameter calibration is effective. Since trips originate from demand, we studied elderly travel behavior from the perspective of travel mode choice, which can accurately determine urban elderly travel characteristics and provide a basis for planning and solving traffic problems in an aging society and, at the same time, provide better service to the elderly for transportation service in China.

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Study on Travel Mode Choice Behavior Using Latent Class Model

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ABSTRACT

Latent class model (LCM) is an analytical technique which is applied to establish model with latent variables. This paper first analyzed the principles of the latent class model, the calibration method of parameters and the evaluation indices after introducing the concepts of latent variables and categorical variables. Then, combined with the number of explicit variables and the number of latent classes which were achieved from the relative fit index, a latent class model of travel mode choice was formulated. Moreover, based on the personal trip survey data of Beijing in 2005, this model was calibrated using Mplus software, at the same time the proportion of explicit variables in the latent variables was obtained. The calibrated parameters were thoroughly discussed, as well as the characteristics of latent classes. The results show that the latent class model provides a more powerful tool to analyze categorical data for the research of travel mode choice problem.

Keywords: travel mode choice; latent class model; latent variable; categorical variable

INTRODUCTION

Mode choice behavior of individuals is driven by many factors including attributes of the mode as well as the social environment and perceptions of travelers which are not directly observed. The aim of this paper is to better understand the underlying choice preferences of travelers and therefore increase the forecasting power of the choice model.

Shen (2009) compared the differences between Latent Class Model (LCM) and Mixed Logit Model on travel mode choice; Bilge et al (2011) integrate latent attitudes of individuals into a mode choice model through latent variable and latent class models with the help of psychometric indicators; Vredin Johansson et al (2006) work on the latent variables of environmental preferences, safety, comfort, convenience and flexibility by analyzing their effects on mode choice with a sequential estimation of latent variable and choice models. Qiu (2008) explained scientifically the principles and techniques of latent class modeling.

Based on the personal trip survey data of Beijing in 2005, this paper introduced personal characteristics and trip characteristics into the influencing factors of travel mode choice, and formulated mode choice models of car and bus using Latent Class Model. The parameters of two models were estimated respectively, and the differences between two models were explored based on the latent class probabilities and conditional probabilities.

THE THEORY OF LATENT CLASS MODEL

Combining the advantages of structural equation model and log-linear model, latent class model (LCM) is characterized by revealing the associations of each manifest variable with least numbers of latent classes until the local independence is achieved.

Latent variables and manifest variables

Latent Variable refers to the one that can't be measured directly, and the state of it must be estimated by statistical approach. Generally, the variable that can be monitored and measured directly can be called manifest, observed or measurable variable.

Latent variables can be defined as either continuous variables or categorical variables. The change of observed variables in the probability will be influenced by the latent class behind it. Therefore, the individual difference of subjects in the latent variables is no longer the differences of degree in the number, but the differences of category in the quality. It can be seen, not only the manifest variables but also the latent variables can be defined in two different forms which includes "continuous" and "category". Therefore latent variable model can be divided into four types, as shown in Table 1.

Table 1. Type of Variables and Its Analysis Method

Latent variables	Manifest Variables	
	category	continuous
category	Latent class analysis	Latent profile analysis
continuous	Latent trait analysis	Factor analysis

If the factor analysis is the best technology to research the continuous latent variables behind the continuous manifest variables, then the latent class analysis is the best technology to investigate the category latent variables behind the category manifest variables. The main difference of latent class analysis and factor analysis is that the form of the variables is "category" or "continuous."

LCM consists of manifest variables and latent variables. The different levels of manifest variables refers to the subjects belonging to different classes when they were measured, however, the different levels of latent variables represents different latent classes after estimate. The value of categorical variables' response probability in each level could be got from that the numbers of different levels is divided by the total number. As for the definition of latent variables, as well as the theoretical relationship between the manifest variables with latent variables will be represented by the way of statistical model which is latent class model. The whole process of analysis, estimate, evaluation, comparison, explaining of the model called modeling.

Probabilistic parameterization

The innovative principle of LCM is converting the probabilities of categorical variables into the model with parameters, that is probabilistic parameterization, which includes two kinds of parameters, one is latent class probabilities and the other one is conditional probabilities. The latent class analysis assumes that every observed data can be classified into one of T clusters of latent variable X, where each cluster is mutually exclusive and independent. The manifest variables measured actually by researchers should satisfy the basic condition of partial independence after considering the potential property of all the observation data. This means that if taking the category properties of observed data into account, the manifest variables should be completely independent and irrelevant. Under the above assumptions, LCM can be represented by the probability function.

If there are three manifest variables A, B, C, LCM formula should contain a un-conditional probability which represents latent classes, showed as π_t^X , and three conditional probabilities which reflected by the manifest variables on the latent class structure consisting of . The most basic latent class model is expressed as:

$$\pi_{ijk}^{ABC} = \sum_{t=1}^T \pi_t^X \pi_{it}^{\bar{A}X} \pi_{jt}^{\bar{B}X} \pi_{kt}^{\bar{C}X} \quad (1)$$

π_{ijk}^{ABC} is a joint probability (the sum of each probabilities) of LCM. Let π_t^X denote the latent category probability When the observed variables are local independent, the probability of latent variable X in the level t , which means the probability of observed object randomly selected from the sample that belong to the cluster t . And the sum of the probabilities of each latent category is one, expressed as $\sum_t \pi_t^X = 1$. The conditional probability such as $\pi_{it}^{\bar{A}X}$ indicates the probability of A at level i under the condition of latent category T, formulated by $P(A=i|X=t), i=1, 2, \dots, I$. Then, the property of conditional probabilities can be formulated as following:

$$\sum_i \pi_{it}^{\bar{A}X} = \sum_j \pi_{jt}^{\bar{B}X} = \sum_k \pi_{kt}^{\bar{C}X} = 1.0 \quad (2)$$

Evaluation indices

The progress of evaluation is to find a model which is simple, with minimum parameters, and better goodness-of-fit. Three indices such as Likelihood Ratio Chi-square Statistics (G^2), Akaike Information Criterion (AIC) and Bayesian Information Criterion (BIC) have been widely used.

1 Likelihood Ratio Chi-square Statistics (G^2)

G^2 is a criterion of model fitting degree, through calculating the difference between the number of observations (f_{ijk}) and the number of expectations ($\hat{f}_{ijk}$) of each cell, and using the logarithmic function to convert to the likelihood ratio chi-square statistic. Expressed as follows:

$$G^2 = 2 \sum_{ijk} f_{ijk} \ln(f_{ijk} / \hat{f}_{ijk}) \quad (3)$$

The value of G^2 will be large if the difference between f_{ijk} and $\hat{f}_{ijk}$ is large. Contrary, if the difference is little, G^2 will be small. It can't be determined that the model proposed fitting measured data best until G^2 is small enough to be considered as a random sporadic value at a certain probability of a wrong decision.

2 Akaike Information Criterion (AIC)

AIC is developed by Akaike based on the information theory, which is used to compare the goodness-of-fit of models. The calculation of AIC is derived by maximum likelihood algorithm. If there are totally H models, $P_h(Y_s)$ is the observation probability of model h.

$$\ln(L_h) = \sum_{s=1}^{2v} n_s \ln[P_h(Y_s)] \quad (4)$$

$$AIC_h = -2 \ln(L_h) + 2m_h = G_h^2 - 2df_h \quad (5)$$

Where df_h is the freedom of model-h. The fitness of a model is best when AIC is the smallest. When comparing two or more models fit degrees, AIC index that reflects the replacement of a new sample for cross-validation of the validity of the likelihood function value increases the amount of the smallest model, so AIC and more used in cross-validated model selection validity. AIC indicators as a standard model fitting degree, tend to choose more efficient one, a simple model, can be understood as the lesser of the estimated parameters, benefit greater freedom.

3 Bayesian Information Criterion (BIC)

The drawback of AIC is that it does not consider the influence of the number of samples. Therefore, the convergence of probabilistic inference is worse when the number of samples get more. Afterwards, Schwarz based on the theory of Bayesian and proposed the bayesian information criterion (BIC) to improve the convergence. The equation is given:

$$BIC_h = -2 \ln(L_h) + \ln(N) \times df_h = G_h^2 - \ln(N) \times df \quad (6)$$

The calculation of BIC includes the adjustment item $\ln(N)$ which take the number of sample into account and benefit to the model having fewer

parameters. When the number of sample reaches thousands of people or the number of parameters is few, we can use BIC to evaluate the model, otherwise the AIC is a better index.

MODEL FORMULATION AND SOLUTION

Model

This article applied two most commonly transport mode in our life to research the behavior of transport mode choice, one is car and the other is bus. The age of travelers, sex, trip distance and trip time are manifest variables. The samples were selected from the personal trip survey data of Beijing in 2005, there were 845 trips by car and 1470 trips by bus. Then classify the data of the continuous variables because the type of manifest variables and latent variables must be categorical variables.

Table 2. The Category of Manifest Variables for Car and Bus

Category	Age		Sex		Distance(km)		Time(min)	
	Car	Bus	Car	Bus	Car	Bus	Car	Bus
1	10-25		Male		0-3	0-8	0-15	0-40
2	25-40		Female		3-6	8-16	15-30	40-80
3	40-55				6-9	16-24	30-45	80-120
4	55-75				>9	>24	>45	120-160

Evaluation of the model

The goodness-of-fit is the best if the evaluation indices are the least. As shown as Table 3, the results of car model, when T (the number of latent classes) =4, apart from BIC other indices are the least. However, BIC is more applicable to the model whose sample reaches thousands of people and the parameters are few, but the two conditions are not satisfied, then do not pay attention to the BIC. In summary, T=4 is the best model.

Table 3. Evaluation Indices

Car			
T	2	3	4 (best)
G ²	285.844	133.741	73.241
AIC	6647.391	6517.288	6477.711
BIC	6746.917	6668.946	6681.502
Bus			
T	2	3	4(best)
G ²	231.454	120.357	99.429
AIC	11835.700	11746.603	11749.198
BIC	11946.854	11915.979	11941.798

As can be seen from the evaluation indices of bus model, when T=4, AIC approximates to the value when T=3, as well as G^2 is the least. BIC values are subject to too large increase in the impact of the model parameters, T=2 with respect to the case of fewer parameters, the fit better.

Parameter estimation results

The last row in the table is latent class probability which denotes the probability of latent variable X in level t. It is determined by two factors, one is the total number of latent classes, denoted by T. Having determined T=4 in two models through calculating the evaluation indices. The other one is the proportion of each latent class, whose size is positively correlated with the importance in the latent variables.

Table 4. Univariate Sample Latent Class Probabilities and Conditional Probabilities

Latent Manifest		Car				Bus			
		1	2	3	4	1	2	3	4
Age	1	0.100	0.119	0.123	0.080	0.293	0.180	0.221	0.353
	2	0.535	0.322	0.272	0.349	0.275	0.351	0.320	0.213
	3	0.285	0.472	0.482	0.531	0.266	0.470	0.338	0.107
	4	0.080	0.087	0.123	0.041	0.165	0.000	0.121	0.327
Sex	1	0.606	0.621	0.701	0.740	0.420	0.188	0.423	0.499
	2	0.394	0.379	0.299	0.260	0.580	0.812	0.577	0.501
Distance	1	0.000	0.459	0.000	0.000	0.000	0.940	0.000	0.933
	2	0.000	0.541	0.569	0.000	0.944	0.060	0.000	0.067
	3	0.000	0.000	0.431	0.453	0.056	0.000	0.647	0.000
	4	1.000	0.000	0.000	0.547	0.000	0.000	0.353	0.000
Time	1	0.010	1.000	0.062	0.000	0.000	0.427	0.000	0.361
	2	0.280	0.000	0.938	0.230	0.630	0.573	0.241	0.639
	3	0.217	0.000	0.000	0.770	0.339	0.000	0.562	0.000
	4	0.493	0.000	0.000	0.000	0.031	0.000	0.197	0.000
Latent Class Probability		0.079	0.163	0.346	0.412	0.418	0.179	0.172	0.231

1. Car

The latent class probability of cluster1 is the minimum. In cluster1, the conditional probability of individuals who are men, aged between 25 and 40 years, trip distance above 9 Km and trip time around 45 minutes is larger than others, it illustrated that the impact of latent class 1 on the manifest variables which meet the above conditions is significant.

The latent class probability of cluster4 is 41.2%. The impact of latent class 4 on the manifest variables which include Age3, Sex 1, Distance 3-4 and Time 3 is remarkable. It represents the individuals who are men, aged between 40 and 45

years, trip time half an hour and distance above 6Km. In reality, middle-aged people more inclined to choose a comfortable way to travel, and the travel speed and the convenient encourage them to choose the car to travel.

Latent class 3 indicates that it needs 15-30 minutes in a short distance traveling by car which just reflects the excessive dependence on the car of Beijing citizens. As the level of economy rising, private cars are becoming more and more common. And at the same time, public transport is lack of comfort and convenience which leads to that people is driving the car even though in a short distance travel. It can be seen that the probabilities of choosing car to travel for men is bigger than women. Besides, the probability of the individuals at the years old 25-55 traveling by car is greater.

2. Bus

The latent class1 is the most significant, which has 41.8%. We can see from the conditional probabilities of cluster1 that the differences of age and gender are not large, but the trip distance concentrated in 8-16km, as well as the trip time concentrated in 40-80 minutes.

Cluster2 represents the group of people who commonly are women, aged between 25 and 55, trip distance within 8km and trip time about an hour. In the real world, compared to women, men prefer to travel by car, which has been corroborated in the above. The reason why 25-55 years old woman travelling in the short or medium distance tend to take bus is related to the psychological quality, the standard of driving as well as some environment factors.

Shown as latent class4, when trip distance is short, the differences of conditional probabilities in sex is not great, however the age of travelers mainly concentrates in 10-25 and 55-75. These people can enjoy minimum bus fare since they generally hold students' cards or olds' cards. The trip cost is very low, so they are inclined to by bus. Through horizontal comparison, it is found that the impacts of latent variables on age and sex do not have great differences.

CONCLUSION

From the above analysis, it can be seen that many residents are over-reliance on the car, and several people would like to travel by bus. In order to reduce urban traffic congestion and encourage people choosing public transport, the government should promote the reasonable and efficient use of private cars, control the number of household cars, improve the standard of public transport services, and at the same time of meeting the travel demands of residents, reduce the dependency on the car as far as possible.

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Evolution and Revelation of a Foreign Road Freight Network Transport

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ABSTRACT

As an advanced cargo organizational structure and mode, road freight network operation can greatly improve the efficiency of road transport and reduce transport costs. With the promotion of the external environment and internal market factors in road freight, developing a road freight network is the direction in China and is an effective way to change the current development situation of China's road freight. Therefore, it is significant to study and learn the evolution of foreign road network transport and development of the law. This study analyzed the evolution process of the development of foreign road freight transport from two angles in the transportation business—outsourcing and the carriage of goods—and summed up the basic laws of foreign road freight transport network development, which is that competition and cooperation are the two main themes. Mergers and restructuring also are important, and contracts and standards are important elements of foreign road freight transport network development. Combined with the characteristics of the transport of a domestic freight transportation network, the focus is on network transport development in China. This will guide enterprises toward market network operation, promote synergies among the main bodies of the freight market, strengthen technical innovation, and guide enterprises to achieve specialization, so that the road freight market can be more efficient.

Key Words: Road freight transport; network transport; evolution; revelation

EVOLUTION OF THE FOREIGN ROAD FREIGHT TRANSPORT NETWORK

The development process of road freight transport services network is mainly due to the impact of three aspects: (1) the stage and structural changes of economic development, more precisely, the form of industrial and product structure, which determine the requirements of road freight transport organization, because meeting consumer demand in a timely manner has become a significant characteristic of road freight transport; (2) the institutional environment of the road transport industry; whether transport resource allocation is determined by the government or is market-driven is a key transport organization deciding factor;

and (3) road transport technological progress, which is closely related to the road freight transport network and is an important support for the development of conditions to a higher form.

The study of road freight transport networks abroad is mainly from two angles: one is based on industry development strategies and is morphological, which is the form and characteristics of the change process of transport company outsourcing; the other is based on the key elements of road freight transport development, including industry institutional environment, transport organization, and information technology and other factors.

DIVISION BASED ON TRANSPORT OUTSOURCING

An important feature of the modern economy is that the scale of outsourcing has greatly increased. Studies show that the annual growth of enterprise outsourcing is 20%. Transport and logistics outsourcing is one of the most common businesses. The impact of transport and logistics outsourcing on the road freight transport industry is primarily that it is a prerequisite for goods aggregation, and transport operations can be released from the transport market, which lays a foundation for large-scale transport. It is the beginning of building relationships of a new type, carrying and consigning and highly efficient partnerships.

From the perspective of transport outsourcing, the development of road freight network transport can be divided into three stages.

Stage 1: Private transport occupied a predominant position before the 1980s. Before then, the relationship between foreign industry and transport development was mainly for two reasons: First was the lack of network transport demand on foreign industrial development. The development of the competitiveness of enterprises had not focused on cultivating core competitiveness and had not tried to reduce the pressures of enterprise operating cost through outsourcing transport and logistics; therefore, most enterprises provided their goods transportation service by owning and renting vehicles. Private transport was dominant, and goods were dispersed by large and small enterprises and had no conditions to realize scale and network operation; meanwhile, due to the small scale, enterprises were unable to support themselves to implement network operations of private transport. Second is the strict economic control in operating routes, scope, etc., which restricted the development of network transport. For example, in the United States, the Motor Carrier Act divided the entire truck industry into route management and regional management; route businesses were required to obtain a business license, and regional businesses had to obtain an area license; the carrier was divided into employed carrier and private carrier, and the employed carrier was divided into common carrier and contract carrier. The regulation was that private transportation carriers generally could not take employed transportation, and the contract carrier was not applicable to general public utilities. Strict industry regulation largely limited the free and reasonable allocation of road freight

transport resources, but also became an institutional factor in limiting the network operation of road freight.

In summary, a company's private goods transport form was an important component of road freight transport. Due to the limits on enterprise size and strict industry control, the degree of development of the road freight network transport was at the lower level.

Stage 2: Beginning in the 1980s, the development of the road freight transport environment changed greatly; third-party network transport services began to take shape. First, outsourcing became very important in economic development, and the outsourcing scale was becoming ever larger. The transport business began to separate from the core business of the company. At this time, professional third-party logistics and transport companies could gather many different manufacturers for sourcing; they merged volume and fully reflected the inherent characteristics of transport economies of scale and economies of scope. Second, transport infrastructure development improved and the road network system was basically formed. For example, the construction of the interstate highway system the United States that began in 1956 was basically completed. Third-party transportation and logistics enterprises depended on the increasingly improved transport infrastructure of country or region in order to construct their own operational network. Third, after 40 years of strict control, developed countries started to relax controls on the road freight industry by reducing tariff levels through competition, optimizing transport structure, and improving network transport efficiency. With the relaxation of freight business routes and the cancellation of business regulation, less-than-truckload (LTL) freight traffic carriers continued to expand their geographic coverage, and a freight business network gradually formed that relied on the freight network to control main sourcing of the market. Meanwhile, it largely dominated the market scale in an increasingly vehicle transportation market.

Stage 3: In 1990s, the development trend of economic globalization demanded the globalization of production and sales and broke down geographical boundaries, industry boundaries, and time limits. Mass production and consumption grew the transportation and logistics scale, and the independent transport service network could not meet the requirements of economic development. Meanwhile, information technology was widely used, not only to promote the service level of network transport, but also to gradually eliminate cargo transport and warehousing, posts, and other industry technical barriers to entry. In road freight network transport development, the phenomenon of industrial convergence was more outstanding, for specific forms: the road freight transport network involved warehousing operations and continued to expand business boundaries and market space to form larger spatial movement of goods and storage services. The road freight transport network was regarded as a platform based on information technology to consolidate resources such as storage, handling, etc., and to improve the capability of an integrated and professional logistics network service.

As the third-party transportation and logistics market gradually matured, a new type of transportation and logistics service provider began to emerge—the fourth-party logistics service provider, which integrated its own and other organizations' resources, capabilities, and technologies and provided complete logistics and transportation services, including transportation and logistics solutions design, organization, and operation. Fourth-party logistics providers and logistics services were the essential elements (including operations networks, relationship networks, information networks, etc.) of road network transport, and could achieve better conformity of transport network operation and integration capabilities.

DIVISION BASED ON FREIGHT

Whether road freight transport development was good or bad depended on the institution, organization, and technology and other basic elements of promotion. Road freight network transport developed different elements and different roles, in which, at a certain stage, there was at least one element that played a leading position. The development stages of the foreign road freight transport network can be divided into three stages.

Stage 1: From the beginning of the 1980s, Western developed countries started to relax economic regulation of road freight transport and began the reform period of transport regulation. The characteristics in this period were toward a more free institution and government retreat from business regulation—for example, the freedom to access markets and set prices. The liberalization measures were not only legal reform, but also actual changes in the interpretation and application of regulations. The United States in 1980 issued the Motor Carrier Act and in 1989 Japan issued the Trucking Business Law to gradually relax economic regulation of the road freight industry. In 1995, the United States ICC Termination Act repealed the federal transportation regulatory agency, the Interstate Commerce Commission (ICC) and eliminated most economic regulations of trucking transportation. At the same time, these measures also transcended national boundaries; for example, the European Community in 1992 explicitly proposed abolishing system obstacles of free transport operation. Lifting economic regulation became a leading factor for promoting the development of a road freight network transport, and the changes in regulation made the transportation supply more flexible and promoted the formation of the road freight network, making it more responsive to small consignments of network operation requirements to reduce costs and enhance the efficiency of road freight transportation.

Another important aspect of the reform of this period was that the content of industry regulation shifted to freight-forwarding transporters and agents and management of other transport organizations. Usually, most car carriers could not afford to maintain transport operations across the country. Real-world carriers in different areas still operated across the country, mainly due to cargo transport

agents in the market. Freight forwarders and agents played a core function in the road freight market network to a certain extent. The United States began to control freight forwarders and agents; for example, freight forwarders and agents needed to register with the STB, purchase cargo insurance, accept responsibility for losses or damage to goods, etc., which not only greatly improved the overall quality of the freight forwarding and brokerage industry, but also contributed to the level of total road freight transport market organization and increased the efficiency of network operations.

Stage 2: The United States issued the Railroad Revitalization and Regulatory Reform Act in 1976, the Airline Deregulation Act in 1977, and the Motor Carrier Act in 1980, which moved U.S. transport into a period of deregulation. The climax was the abolishment of the Interstate Commerce Commission and trucking regulation in 1995. Deregulation eliminated the obstacles of comprehensive transportation development and promoted the growing and maturing of the organization of multimodal transport and “drop and pull” transport. In 1986, the Truckers Association Agreement allowed a shift of up to 28% of long-distance LTL goods transport to comprehensive transportation. Between 1980 and 2002, United States rail intermodal volume (including the transport of highway trailers and containers on a flatcar) increased three times; the number of trailers and containers grew from 3.1 million to 9.3 million.

During this period, the wide use of containers and the general development of “drop and pull” transport organizations not only improved the exchange efficiency of road freight and the overall operation efficiency of the network, but also reduced the possibility of damaged goods and improved the quality of transport service. The outstanding contribution of transportation organization innovation-led network transport development was that it greatly promoted the alternative merger of road freight network transport in rail, waterways, and aviation networks and formed a network of goods intermodal transport, thereby lowering costs and increasing competitiveness.

Stage 3: Into the 1990s, the scale of goods movement and demand characteristics had changed greatly. One change was that the trend of economic globalization was already evident; the global flow of goods was more frequent and the flow scale was also growing. The second was the trend to miniaturization but more frequent private delivery, and the larger proportion of goods directly reached the end user. Third, enterprises pursued better performance of the supply chain, often emphasizing that transport should have the high efficiency requirements of an express courier. Road freight transportation development was difficult to adapt to the above requirements based on traditional ways. Therefore, as the scope of goods widened and the transport scope expanded, road freight development relied more on information technology progress and support through the use of modern information and communication technologies and promoted network transport services, where it could greatly reduce the transaction and management costs of goods network operation.

The application of information technology innovation greatly improved the

global express network. After decades of development, Western countries had emerged with a large number of express cargo transport network giants, such as Federal Express and UPS (U.S.), DHL (Netherlands), TNT (Australia), and so on. These express freight companies operated a large termination, pickup, delivery, transport, and long-distance transport network and were able to successfully adapt to the global trend of logistics development. One important reason was the active adoption of modern science and technology and the application of modern communication, information, and network technologies to completely renovate and upgrade the technical capacities of the enterprise express network. A typical enterprise was Federal Express, which, in 1973, was the world's largest express transportation company. Its main operation characteristic was fully utilized electronic information and network technology.

BASIC LAW OF FOREIGN ROAD FREIGHT TRANSPORT NETWORK DEVELOPMENT

Competition and cooperation were the two main themes in the development of the road freight transport network. As the scope of goods transport constantly expanded, it was more and more important for road freight enterprises to have an excellent transportation network. Shippers generally did not need so many transport service providers; they were willing to select carriers from enterprises with a perfect transportation network. Hence, network expansion and improvement became a significant element of competition in the freight transport market. By expanding and strengthening the transport networks of different transport enterprises from developed countries, the theme always was competition and cooperation. For shippers with smaller and more decentralized cases, small-scale carriers participated in the competition in that their transport network was usually compact, had a limited scope, could support a highly efficient operation, and had certain administrative advantages and cost advantages. Therefore, small-scale carriers were able to compete with large-scale carriers. With the wide application of information technology, the resource integration and market organizational capabilities of the large-scale enterprises were further enhanced and their coverage extended further, so the market share of small firms was occupied. Meanwhile, the appearance of a shipper's union caused sources to concentrate on large enterprises with perfect networks. As such, small firms took chances to cooperate with large enterprises and became their agent carriers.

Whether in today's competitive market structures or the future's sustainable development requirements, greater cooperation in the network is inevitable. Research in the EU indicates that the direct partnership is dominant in the market, with more than half of companies adopting network partnerships and less than 40% of companies adopting loose partnerships. All of these partnerships, to a certain extent, ensure a stable and lasting partnership.

The progress from competition-dominated to cooperation-dominated is a basic law in the development of road freight transport in developed countries and

marks the emergence of a new model of network development—a freight market network of enterprise management.

MERGER AND RESTRUCTURE IN FOREIGN ROAD FREIGHT TRANSPORT NETWORK DEVELOPMENT

Since the 1980s, shippers strategically developed through mergers and alliances to form a larger shipper network and to further enhance control of the supply chain. While shippers increasingly merged, the road freight industry as the market supplier was still in a split state. To achieve a balance with shippers, the road freight industry began extensively to merge and restructure and the market concentration was constantly increased. A service network was formed that covered the country and extended overseas.

After the United States launched an open market policy in 1980, 66 of the top 100 road transportation enterprises disappeared from 1982 to 1993; 47 went bankrupt and merged with the top 100 road transportation enterprises. According to statistics, in 1999, the number of mergers and acquisitions among United States logistics and transport companies was 23, and the total amount reached \$625 million. In 2002, the United States had 2,345 freight companies to go bankrupt; their original market shares were quickly merged and split by other shipping companies.

CONTRACTS AND STANDARDS AS SIGNIFICANT WAYS OF FOREIGN ROAD FREIGHT NETWORK TRANSPORT DEVELOPMENT

The development process of a road transport network was a process of deepening teamwork, and the vital means of cooperation was signing contracts among enterprises, which would guarantee the stability of the enterprise network, and ensure a smooth functioning of the logistics network and prevent the risk of network outages caused by an unstable market. Usually, there are four contract forms in the transport market: a framework contract, a fixed commercial contract, an agreement, and a letter of intent. Among these, the framework contract provides a legal basis for cooperation, and the specific terms are determined through negotiation. A fixed commercial contract includes all details in the contract and has a more legal basis. An agreement is a relatively flexible form, and a letter of intent simply expresses willingness at the beginning of cooperation. According to the survey of the EU, in all these contract forms, the agreement is dominant, followed by the framework contract and the fixed commercial contract.

A uniform standard is another important basis of effective operation of foreign road freight transport network. In international containers and the development of electronic data interchange technology, each country has further implemented international standards as trading conditions in transport and technical equipment specifications, especially in documents, the legal environment, management methods, and other aspects, promoting domestic

transport and international transport merging with each other. Developed countries basically form a mature and perfect transportation standard system that has become a vital way to improve network efficiency, effectiveness, and competitiveness.

GUIDANCE FOR CHINESE ROAD FREIGHT NETWORK TRANSPORT DEVELOPMENTS

China and developed countries have experienced several stages of economic development; from the perspective of outsourcing, transport experienced three stages, including private transportation, third-party network transportation services, and the present conformity and integration of network transport services. At present, a decentralized mode of transport is still dominant in the Chinese road freight market, with network structure and mode present only in LTL freight, small freight, and similar areas; network performance has not formed a scale enterprise that urges industry to develop. Overall, the level of the road freight network in China is relatively low. Therefore, we need to fully learn from the development features of other road freight networks and follow the transportation laws of economic development to promote a freight network to a higher level.

In contrast to abroad, at present, the development key for China's road freight network transport is to guide enterprises in philosophy toward market network operation and promote synergies between the main bodies of the freight market, strengthen technical innovation, and guide enterprises to achieve specialization, so that the road freight market can be more efficient.

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Research on Public Transportation Development Model Selection in New Urban Areas

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ABSTRACT

New urban districts are the inevitable outcome of urban development at a certain stage, and good or bad planning directly affects the development of the whole city. The choice of a public transportation development mode is an urgent problem in the process of the implementation of public transport priority development in urban districts including the functional orientation, traffic characteristics, and current situation and problems related to land use, combined with existing technical and economic characteristics of the city's main public transit mode. This paper probes the influence factors of public transit mode choice and establishes a new urban district public transit mode decision matrix. Finally, taking the Nanjing Hexi new district as an example, the paper verifies the applicability of the matrix.

Keywords: Urban district, Public transit, Mode selection

INTRODUCTION

Since China's reform and opening up, industrialization and urbanization have been continuously accelerated. To adjust the industrial structure of cities, it is necessary to transfer or transform the original industrial enterprises. In the meantime, it is necessary to develop new economies and improve the living standard of residents by introducing new high-tech industries. The need for new space promotes the expansion of the city area, which generates a great number of new area development activities. Currently, transportation in the new areas of many cities in China is not convenient, especially when public transportation development falls behind. If this continues, city residents will likely choose individual transport, which may give rise to new problems and does not benefit the development of new urban areas. As an organic part of cities, new areas need a convenient and resource-saving model of public transport. They vary in their states, scales, and functions, as do the transport structures that serve them. This paper researchs a public transportation development model to support and guide the construction and development of new urban areas.

PUBLIC TRANSPORTATION DEVELOPMENT SITUATION IN NEW URBAN AREAS

Transportation characteristics and land use of new urban areas.

(1) Transportation characteristics of new urban areas: Due to differences in functions and space structures, new areas have different transportation characteristics. New areas can be divided by different functions as follows: 1) New areas with major functions of residence and education: these have large demand on passenger traffic, and city residents usually travel for work, school, etc., which results in the tidal current characteristics of new urban areas. If there is no established or complete public transportation system, the individual transport ratio will increase gradually; 2) New areas with major functions of industry and retail: these areas have high demand on goods traffic service. This indicates that the tidal current characteristics and public transportation are mainly reliant on normal public ones; 3) New areas with the major function of tourism: these have large passenger capacity and demand for a good quality of transportation service. Rapid transport or mass transit can be constructed according to the practical situation.

(2) Land use of new urban areas: Land use in cities falls into two categories: low-density dispersion type and high-density centralization type. The former leads to separate functions and low-density development but the latter leads to cascading functions and high-density use. Generally, the high-density centralization type should develop a suitable high-density public transportation model and the low-density dispersion type moves towards a free transportation model. To date, most cities in China have a high-density centralization type. Therefore, a public transportation model should be developed to match with the land use in new urban areas.

Development lag of public transportation in new urban areas. Overseas, countries have already started research on the development of public transportation models in new urban areas. There are two model categories: an MRT model and a bus or tramcar model. New area planning in Singapore and Japan is representative of the former, while Budapest is representative of the latter.

China initiated new urban area planning and public transportation construction in the 1990s. In terms of practical operation, a public transportation system has deviated from the developing direction of a new urban area. Some public transportation systems of new urban areas have lagged behind the developed areas. The transportation development of new areas is not vibrant and is disconnected with the central city. The public transportation in new areas without their own planning is too reliant on the resources of the old developed areas, since transportation construction in most of the new areas of China is oriented by regional economy growth points. All these will exert a great influence on future development.

INFLUENCING FACTORS FOR THE SELECTION OF PUBLIC TRANSPORTATION MODELS IN NEW URBAN AREAS

Site selection and lay out and of new area and its types. Development of a new urban area is based on site selection and layout. It is not reliant on the old developed area and develops itself independently. Its public transportation model also shows related characteristics.

Land use of new urban areas. Related to transportation, land use is an indication of the socioeconomic activities in the space. If there is a need to improve the service quality and effectiveness of public transportation with limited land resources and the coordination of public transportation and land use, the selection of the mode of public transportation controls the progress of land use. In terms of predominance, development models can be divided into three categories: land use predominant model, public transportation predominant model, and model of land use and public transportation developing together.

Functions of new urban areas and its future development. To fulfill the major functions of new urban areas, economic growth aspects must be developed to match and promote the development of the new urban areas at the early stages of construction. For example, new areas with major functions of residence and education have a large demand on passenger traffic, and city residents usually travel for work, schools, etc., which will result in tidal current characteristics of new urban areas. If there is no established public transportation system, the individual transport ratio will increase step by step. New areas with the major function of the manufacturing industry have normal passenger capacity and need gradually normal public transportation. New areas with the major function of tourism have large passenger capacity and high demand for a good quality of transportation service. Rapid transport can be added appropriately to better serve the passengers. New areas with major functions of industry and retail have a high demand for goods traffic service. This indicates that the tidal current characteristics and public transportation are mainly reliant on normal public ones.

Scale of new urban areas. The scale of new areas and their economic power have a great influence on public transportation models in new urban areas and their central cities. In general, regarding the construction area and population, internal passenger capacity is less than that of central cities with a strong economy. Normal transport is referred to as regular public transportation mode. For bigger scales, rapid transport can be the auxiliary mode.

RESEARCH ON OPTIMIZED SELECTION OF PUBLIC TRANSPORTATION MODEL IN NEW URBAN AREAS

Establishment of evaluation index system. According to the influencing factors for theselection of a public transportation model in new urban areas, a comprehensive evaluation system should be established following the principles of science, simplicity, operability, integrity and comparability. Details are shown in Figure 1.

Model establishment. Selection of a public transportation model in new urban areas is a multi-attribute value decision. All indices might be contradictory to one another. Therefore, the matter of choice is described as follows:

$$DR\{f_1(\bar{x}), f_2(\bar{x}), \dots, f_n(\bar{x})\} \quad (1)$$

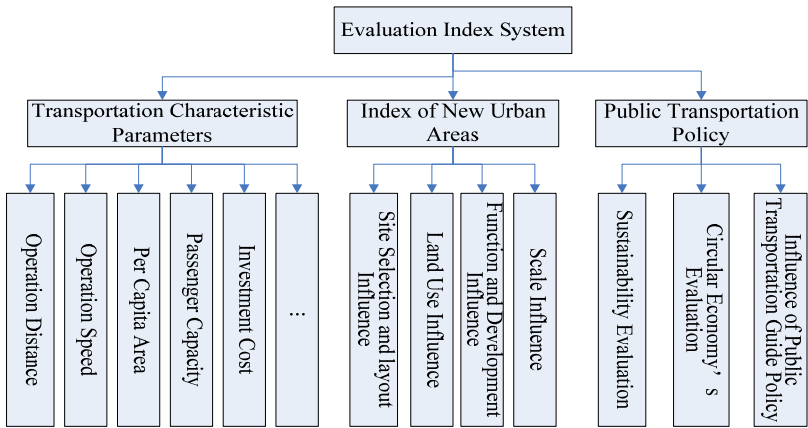


Figure 1Evaluation index system

Where DR = appropriate decision rules; $\vec{x}$ = decision vector= $(x_1, x_2, x_3, x_4)^T$, x_1 = normal transit, x_2 = rapid transit, x_3 = LRT, x_4 = MRT....., and $f(\vec{x})$ = target vector.

In Eq. (1), selection of a public transportation model in new urban areas can be described as finding the optimal public transportation model in a new urban area from the feasible solution group x by using appropriate decision rules and determining the most optimized solution among the group. The process of choosing the matrix attribute values is shown in Table 1.

Table 1. Choice Attribute Value

	Index of Characteristics	Normal Transit	Rapid Transit	LRT (x ₃)	MRT (x ₄)
1	Suitable operation distance(km)	2~10	2~15	3~20	4~30
2	Operation speed(km/h)	15~25	20~30	20~40	30~40
3	Per capita area (m ² /per person)	0.5	0.37	0.4	0.26
4	Passenger capacity (10000 person-time/h)	1	6	3	7
5	Investment cost (million RMB/km)	5~15	20~100	100~200	300~800
6	Accessibility	Fairly Poor	Fairly Poor	Poor	Poor
7	Conformability	Fairly Good	Fairly Good	Good	Good
8	Safety	Fairly Good	Fairly Good	Good	Good
9	Sustainability evaluation				
10	Circular economy's evaluation				
11	Influence of public transportation guide policy				

12	Site selection and layout influence of new area
13	Land use influence of new area
14	Function and development influence of new area
15	Scale influence of new area

Regarding the selection of a public transportation model in new urban areas, a choice matrix should be set up. Due to the differences between public transportation both inside the new urban area and that of connecting new areas with the central city, the evaluations of Indexes 11–15 differ in some aspects.

(1) *Processing of quantitative index:* With the influences of objective factors, every quantitative index converts between advantages and disadvantages. The optimal value is 1, and the worst value is 0. If a quantitative index is set to a certain range, then we obtain the result of z_{ij} for any input y_{ij} within the range between $y_j^{\max}$ and $y_j^{\min}$. For profit-oriented data (such as operation distance, operation speed and passenger capacity), z_{ij} can be computed using the following equation:

$$z_{ij} = \frac{y_{ij} - y_j^{\min}}{y_j^{\max} - y_j^{\min}} \tag{2}$$

For cost-oriented data (such as per capita area and investment cost), z_{ij} can be computed using the following equation:

$$z_{ij} = \frac{y_j^{\max} - y_{ij}}{y_j^{\max} - y_j^{\min}} \tag{3}$$

(2) *Processing of qualitative index:* A qualitative index mainly includes the policy index of the public transportation model and the adaptability index of the new urban area. There is no related detailed data to support, so scores by experts are processed.

(3) *Analytic weight process of attribute value index:* Selection of a public transportation model in a new urban area is a matter of a multi-attribute value decision. All index weights have influence on the objectivity of this matter. This paper applies Analytic Hierarchy Process (AHP) to define X_i ($i=1, 2, 3, 4$), the weight index of the public transportation model in new urban areas, and simplifies the calculation by AHP via EXCEL.

Process and Analysis of Attribute Value of Choice Matrix

After the preliminary processing of the quantitative index, Results of Quantitative Index after [0, 1] conversion are shown in Table 2.

Table 2. Normalization Processing of Quantitative Index

Index of Characteristics		Normal Transit (x ₁)	Rapid Transit (x ₂)	LRT (x ₃)	MRT (x ₄)
y ₁	Operation distance(km)	0.00	0.23	0.50	1.00
y ₂	Operation speed(km/h)	0.00	0.33	0.67	1.00
y ₃	Per capita area(m ² /per person)	0.00	0.54	0.42	1.00
y ₄	Passenger capacity (10000 person-time/h)	0.00	0.83	0.33	1.00
y ₅	Investment cost(million RMB/km)	1.00	0.91	0.74	0.00

Evaluation of Indexes 6–10 is applicable to public transportation both inside the new urban area and the new areas connecting with the central city. Here in, this is referred to as basic qualitative index; however, due to the differences of new urban areas, Indexes 11–15 cannot be uniformly disposed, so they are classified and analyzed here. The converted data is shown in Table 3.

Table 3. Processing of Qualitative Index

Index of Characteristics		Normal Transit (x ₁)	Rapid Transit (x ₂)	LRT (x ₃)	MRT (x ₄)
y ₆	Accessibility	1.00	0.68	0.10	0.00
y ₇	Conformability	0.00	0.37	0.80	0.80
y ₈	Safety	0.00	0.32	1.00	0.88
y ₉	Sustainability evaluation	0.30	0.78	0.90	0.83
y ₁₀	Circular economy’s evaluation	0.30	0.76	1.00	0.90

Determination of choice matrix of public transportation model of new area.

After calculation of Attribute Value and Attribute Value Index’s weight, the advantages and disadvantages of the public transportation development model *i* (*i*=1, 2, 3, 4) are sequenced in order in accordance with the Value of *C_i*.The maximum is 1 and minimum is 0. The higher the value, the better. Then, in a conclusion of sequencing of all the public transportation models, a suitable model can be determined.

$$C_i = \sum_{j=1}^n X_j z_{ij}$$

(4)

The optimal public transportation development model in the new urban area is computed, and then it is necessary to verify the chosen model. This paper verifies the suitability of the chosen model by satisfying the maximum peak passenger capacity per hour of the passenger corridor in this area. It distinguishes the balance between supply and need of the public transportation model in the new urban areas through definition of a match index of the public transportation model in the new urban areas. The calculation method is as follows:

$$E = \frac{V}{K} \tag{5}$$

In the formula, E = supply and need match index of the public transportation model in the new urban areas; V = passenger capacity of the public transportation model in the new urban areas; K = current maximum peak passenger capacity per hour of the passenger corridor in this area. The value of E related to supply and the need of the respective public transportation model is as follows:

$$\left\{ \begin{array}{l} E \leq 0.9 \\ 0.9 < E \leq 0.95 \\ 0.95 < E \leq 1.05 \\ 1.05 < E \leq 1.10 \\ 1.10 < E \end{array} \right.$$

APPLICATION EXAMPLE

The Nanjing Hexi new area is an important part of the “one city, three areas” development strategy. The public transportation mode choice is the premise of a Hexi new city to realize TOD mode development. The Attribute Value was not calculated because the planning of a Hexi New Town Area is not guided by a public transportation model. This paper determines only the quantitative index without repetitive calculation according to experts’ scores on Attribute Values. We studied the public transportation model of a Hexi New Town Area and gave raw scores on Attribute Values. Results are shown in Table 4. Subsequently, data in Table 4 were converted according to the 0-1 standard. The best and worst Attribute Values of every item are 1 and 0, respectively.

Table 4. Preliminary Processing of Attribute Value of Choice Matrix of Public Transportation Model of Hexi New Town Area

	Index of Characteristics	Normal Transit	Rapid Transit	LRT	MRT
1	Operation distance(km)	6	8.5	11.5	17
2	Operation speed(km/h)	20	25	30	35
3	Per capita area(m ² /per person)	0.5	0.37	0.4	0.26
4	Passenger capacity (10000 person-time/h)	1	6	3	7
5	Investment cost (million RMB/km)	10	50	150	550
6	Accessibility	1.00	0.68	0.10	0.00
7	Conformability	0.00	0.37	0.80	0.80
8	Safety	0.00	0.32	1.00	0.88
9	Sustainability evaluation	0.30	0.78	0.90	0.83
10	Circular economy’s evaluation	0.30	0.76	1.00	0.90
11	Site selection and layout influence of new area	0.85	0.20	0.30	0.80

12	Land use influence of new area	0.78	0.40	0.80	0.90
13	Function and development influence of new area	0.70	0.40	0.97	0.84
14	Scale influence of new area	0.70	0.83	0.60	0.20

Subsequently, the Attribute Value Index of the public transportation model of the Hexi New Town Area is decided by AHP. The results are shown in Table 5.

Table 5. Attribute Value Weight

Index	1	2	3	4	5
Weight	0.0213	0.0197	0.0493	0.0213	0.0982
Index	6	7	8	9	10
Weight	0.0670	0.0530	0.0713	0.0530	0.0430
Index	11	12	13	14	
Weight	0.0722	0.0218	0.1564	0.2029	

Public transportation model selection and verification. Calculation on the basis of the Attribute Value and Attribute Value’s Weight mentioned above is as follows:

$C_1 = 0.7062; C_2 = 0.5328; C_3 = 0.7280; C_4 = 0.5680$

According to comparison based on the data sequence, it is appropriate to establish LRT on the passenger traffic corridor inside the Nanjing Hexi New Town Area as well as the complete common traffic network in other areas. Due to the limited length of this paper, we do not testify to the effectiveness of the results.

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Development of Resource Allocation Strategies for Operating and Maintaining Traffic Signal Systems

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ABSTRACT

Functional vehicle detection and signal system communication are essential for a traffic signal system to execute its intended coordinated signal timing plans. The degradations of vehicle detection and signal system communication could impose barriers to successful operations of a traffic signal system. To achieve and maintain an acceptable operational level set by a traffic agency, an agency has to prioritize its resource allocation. This paper first introduces the concept of traffic signal system performance. It defines a traffic signal system degradation index, which measures the proportion of increased travel time attributable to failures in detection and communication components with respect to the total observed travel time of a traffic signal control section. The paper uses the degradation index and traffic simulation to evaluate the effectiveness of major strategies to improve vehicle detection and signal system communication. These strategies include upgrading detection technologies, improving the responsiveness to detection and communication repair work, providing adequate training, increasing the frequency of inspection, and installing a GPS clock to maintain synchronization of the time clock in the controller. Finally, recommendations for optimal allocation of operation and maintenance resource allocation strategies for traffic signal systems are provided.

DEGRADATION OF TRAFFIC SIGNAL SYSTEM COMPONENTS

The degradation of a traffic signal system performance comes not only from the outdated traffic signal timing but also from the degradation of its physical system. Two major sources of degradation of a physical signal system are degradation of the vehicle detection component and the system communication component. Functional vehicle detection and signal system communication play important roles for a traffic signal system to execute its intended coordinated signal timing plans. To achieve and maintain an acceptable or target operational level (e.g., 95% of detectors are

functional and 90% of traffic signal systems are online), traffic signal system management must prioritize the resource allocation such that the overall system goals are met.

CONCEPT OF TRAFFIC SIGNAL SYSTEM PERFORMANCE

Performance measures observed in the field such as travel time and travel delay contain the combined effect of physical system status and existing signal timing under prevailing conditions. The numerical values of the observed performance measures cannot completely reflect traffic signal system performance itself because the same observed performance can result from numerous combinations of physical system status, existing timing, and traffic conditions.

Under ideal conditions, with a fully-functional physical system and updated signal timing plans, the traffic signal system can achieve its ideal performance, which generally represents the best performance of the system under a given prevailing condition. The proposed concept of traffic signal system performance considers the potential degradation of a physical system and traffic signal timing, as shown in Figure 1. Any departure from ideal conditions, whether due to degradation of the physical system (detection, communication, and/or others) or timing plans, will cause a gap in performance measure between the ideal condition and the current conditions with degradation, as shown in Figure 1.

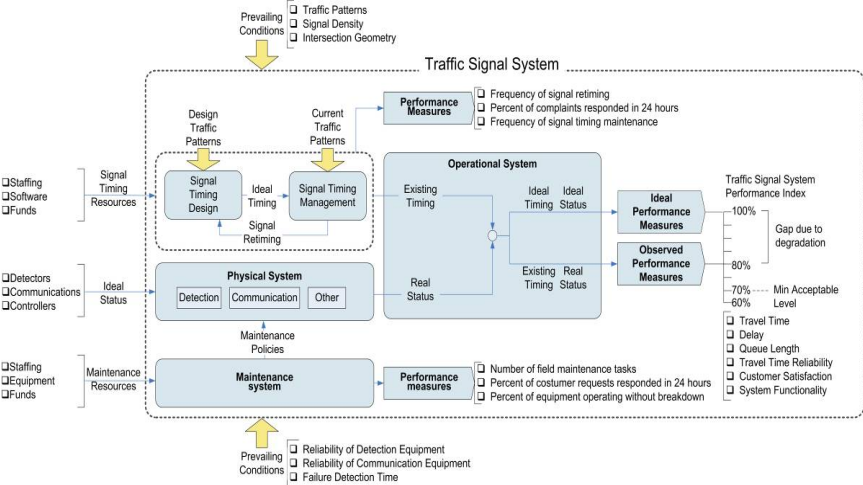


Figure 1. Concept of Traffic Signal System Performance Considering Degradation of Physical System and Traffic Signal Timing.

TRAFFIC SIGNAL SYSTEM DEGRADATION INDEX

Travel time is one of the most commonly-used performance measures for corridors. To adequately evaluate the performance of a traffic signal system itself, this research defines a Degradation Index (DI) of a traffic signal system as the percent of increased

travel time due to traffic signal system degradation relative to its observed travel time. “Ideal travel time” is the travel time the system can achieve under fully-functional detection and communication systems and ideal signal timing, usually through signal retiming. The relationships among observed travel time, ideal travel time, and increased travel time are shown in Figure 2.

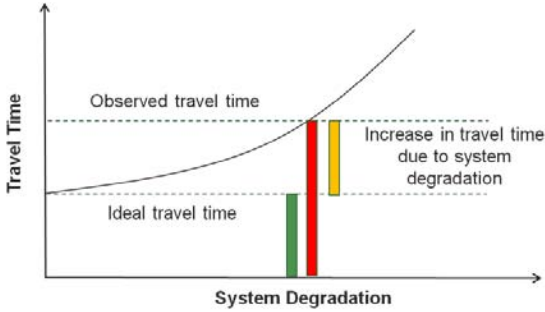


Figure 2. Degradation Index with Respect to Travel Time.

The DI can be expressed as follows:

$$DI = \frac{TT_{increased}}{TT_{Observed}} \times 100\%$$

Where

DI: Degradation Index, in percent (%)

TT_{increased}: Travel time increased due to system degradation

TT_{observed}: Observed travel time under existing conditions

DEVELOPMENT OF RESOURCE ALLOCATION MODELS

Resource allocation performance models. A generic signal control section is a common element of all traffic systems. Performance-oriented resource allocation models based on a generic signal control section were developed in this study. To develop these models, traffic simulation was used as a tool to assess the effectiveness of proposed improvement strategies on alleviating detection and communication degradation with respect to the improvement of DI. The proposed resource allocation model allows the traffic signal system management to have adequate information to allocate the amount of resources applicable to detection and communication maintenance to achieve the desired operational goals.

Overview of the effect of resource allocation in system performance. The components of a traffic signal system suffer breakdowns for a variety of reasons, such as equipment failure, environmental conditions, and installation procedures. Component failure can be characterized by the frequency and time it takes for repair. The time between the start of component failure and completion of repair is usually referred as downtime. Changes to equipment with better technologies will have an effect on the frequency of failures, whereas improving maintenance operations will reduce the downtime of the components.

Evaluation of improvement strategies. The degradation of communication and detection components can translate into an increase in travel time. The degree of the degradation can be expressed by DI. The DI can be used in conjunction with the failure rate of system components and their associated downtimes to obtain a more comprehensive measure of total increased system travel time. For example, the hour that best represents the traffic flow of a control section during a day is 300 vehicles per hour per lane (VPHPL). The observed travel time for the corridor is denoted by T, and the quantity of (DI × T) represents the increased travel time due to existing degradations to detection and/or communication components in the control section. The improvement strategies presented in this paper are oriented to minimize the impact of detection and communication degradations. A discrete-event simulation approach was taken in this study to evaluate the effectiveness of the studied strategies on alleviating detection and communication degradations with respect to the improvement of DI.

Strategies to improve detection. The following section proposes and evaluates several major improvement strategies that can be adopted to decrease the impact of detection degradation in traffic signal system operations.

Upgrade detection technology. Inductive loop is the detection technology most used by agencies, followed by video detection and microwave. Minor detection malfunctions can be fixed by resetting the detector inside the cabinet or by performing minor repairs. According to an agency survey in Florida, this can be done in less than three days for both inductive loop and video detection. On the other hand, major failures require a significant amount of time, especially for loop detectors. Most agencies in the survey indicated that it takes more than 15 days to repair broken loop detectors.

This situation reflects ongoing practices that include contracting out the detector repair work. The expected life of detection equipment is 10 years for both loop and video detection. Using this information, a discrete-event simulation model was built to evaluate the average degradation index per year for a typical control section. The simulation consisted of a total of 180 scenarios. For each scenario, detector failures were simulated according to their life expectancy and downtime, measuring the traffic signal system degradation index per day. A summary of the results is presented in Table 1.

Table 1. Average Degradation Index for Vehicle Detection Type.

Detection Type	Average Downtime of Major Detection Failures (Days)	Average Degradation Index		
		300 VPHPL	400 VPHPL	500 VPHPL
Loop	19.04	5.27%	2.71%	0.72%
Video	7.705	5.54%	2.98%	1.00%
Microwave	10.00	5.55%	2.99%	1.00%

The percentage of reduction in the impact on travel time due to degradation of detection equipment is presented in Table 2. Note that this comparison is for the current situation based on survey data.

Table 2. Percentage of Reduction on Increased Travel Time Due to Detection Failures by Using Non-Intrusive Detection.

From	To	300 VPHPL	400 VPHPL	500 VPHPL
Loop	Microwave	55%	52%	41%
Loop	Video	78%	77%	71%
Microwave	Video	51%	51%	50%

Under equivalent conditions of expected life and downtime, video detection and microwave detection behave in a similar fashion. Inductive loops will exhibit more failures than cameras since the number of loops is two or more times greater than that of cameras for a typical control section. This relatively large number of units increases the chance for loop detectors to fail and cause degradation.

Reduction in detector downtime by improving responsiveness of contractors. From the agency survey, it was determined that 57 percent of the respondents contract out 75 percent or more of their detection repair work, which is mostly inductive loops. When repair work is contracted out, there is an additional downtime due to paperwork and negotiations with the contractors. Reducing the amount of time to complete a detection repair job could be a cost-effective policy, especially for agencies relying heavily on loop detection. As an example, a policy consisting of completing a job order within 3–7 days (5 days on average) was simulated. Table 3 presents the percentage of improvement if the repair time of detection work could be shortened from the current situation (19.2 days on average) to 15, 10, and 5 days of downtime, on average.

Table 3. Percentage of Reduction on Increased Travel Time Due to Detection Failures by Improving Responsiveness.

From	To	300 VPHPL	400 VPHPL	500 VPHPL
Loop-Current	Loop-15	22%	22%	22%
Loop-Current	Loop-10	46%	46%	46%
Loop-Current	Loop-5	71%	71%	71%
Video-Current	Video-5	43%	43%	43%
Microwave-Current	Microwave-5	44%	44%	44%

Increase detector uptime by providing adequate training and procedures for installation of detectors. The nominal value for the life of video detectors and loop detectors with proper installation and regular maintenance is 10 years. An indirect assessment of the effect of training and inspection of a suitable range for the equipment life varying from 2–10 years was divided into three equally-spaced

intervals (low, medium, and high). In the same fashion, three levels of training were determined (low, medium, and high). It is logical assume that better training and inspection help to achieve the maximum life of equipment. The results of the simulation test on performance are presented in Table 4. It can be observed that loops have more potential to be affected by training and inspection than microwave and video detection technologies.

Table 4. Percentage of Reduction on Increased Travel Time Due to Detection Failures via Improvement in Training and Installation Inspection.

Detection Technology	From	To	300 VPHPL	400 VPHPL	500 VPHPL
Loop	Low	Medium	56%	56%	56%
Loop	Low	High	71%	71%	71%
Loop	Medium	High	34%	34%	34%
Video	Low	Medium	53%	53%	53%
Video	Low	High	72%	72%	71%
Video	Medium	High	40%	40%	39%
Microwave	Low	Medium	62%	62%	61%
Microwave	Low	High	76%	76%	76%
Microwave	Medium	High	35%	36%	38%

Strategies to improve communication. The traffic management center can usually communicated with coordinated signals with by different communication technologies, ranging from dial-up to wireless communication. Central control typically has the highest communication requirement among the different control architectures. Time-based architectures present some elementary form of communication that may not be able to generate status reports. Communication can help to reduce the downtime of detection problems by enabling system operators to run status checks periodically. Communication failures are usually translated into loss of coordination due to clock drifting. The following section includes a list of actions seeking to minimize the impact of communication failures on the performance of the traffic signal system.

Increase responsiveness in repair time. A simulation of different response times from 3–20 days and the corresponding lost time during the downtime period was developed. The simulation result showed that communication failures also affect high traffic volumes. This effect was not observed in the detection failure experiments since the advantage of actuated control decreases at higher volumes. Table 5 presents the percentage of improvement if the repair time of communication links can be shortened, whether in-house or through an external contractor.

Table 5. Percentage of Reduction in Increased Travel Time Due to Communication Failures by Improving Responsiveness in Repair Time.

From	To	300 VPHPL	400 VPHPL	600 VPHPL
20 days	15 days	32%	36%	42%
15 days	10 days	41%	45%	52%
10 days	5 days	41%	60%	70%
5 days	3 days	43%	46%	56%

Increase frequency of inspection. The performance of twisted-pair and leased-line systems often can be improved by periodic maintenance of system reports and some simple diagnostic tests to reduce system downtime. As a measure of system performance, the effects of different frequencies of inspection were simulated. Table 6 presents the results of improving the frequency of inspection. It can be observed that for congested volumes by running communication checks every 15 days, the increase in travel time due to communication failures is greatly reduced.

Table 6. Percentage of Reduction on Increased Travel Time Due to Communication Failures by Improving the Frequency of Inspection.

From	To	300 VPHPL	400 VPHPL	500 VPHPL	600 VPHPL
30 days	15 days	62%	66%	75%	100%
15 days	5 days	67%	71%	81%	-

Install time-of-day receivers (GPS clock). When communication fails, if the time-of-day is not maintained with the required precision, the relative offsets of the controllers will begin to drift, causing a loss of coordination. To alleviate this situation, vendors have supplied external electronic clocks that are installed in each intersection cabinet. Generally, the clocks receive time-of-day broadcasts from the NIST's WWV time service or from GPS satellites. The clocks generally output a pulse once per day to reset the controller's time-of-day clock to a specified time. This will keep the intersections in a control section synchronized in the event of a communication failure.

Recommendation for Resource Allocation

The information for each recommended improvement strategy is valuable for traffic operations practitioners. To determine a practical method for allocating maintenance resources for traffic signal systems, the following simple steps are recommended:

1. Determine the cost for performing each improvement strategy considered.
2. Determine the benefit (e.g., travel time reduction) of each improvement strategy considered. The benefits can be computed from the tables of percentage of reduction in increased travel time for selected improvement strategies presented in the previous section.

3. Compute the benefit per dollar invested in the improvement strategy, based on the information obtained from Steps 1 and 2.
4. Rank the improvement strategies, based on the results from Step 3.
5. Select improvement strategies, based on the needs for the traffic signal system and available maintenance resources of the agency.

CONCLUSIONS AND RECOMMENDATIONS

The major conclusions from this research are as follows:

1. Detection and communication component failures significantly affect the operation of a traffic signal system.
2. Traffic signal performance should measure the performance of the traffic signal system itself and include the impact on system degradation.
3. The recommended strategies to effectively improve vehicle detection include upgrading detection technologies, reducing detector downtime by improving the responsiveness of the contractors, and increasing the detector uptime by providing adequate training and procedures for installation of detectors.
4. The recommended strategies to effectively improve signal system communication include increasing the responsiveness in the repair time, increasing the frequency of inspection, and installing a GPS clock to maintain synchronization of the timeclock in the controller.
5. The performance and benefits of the recommended improvement strategies are included in this paper.
6. Transportation agencies can use the process presented in this study to compute the percent of improvement per dollar invested in the improvement strategy. After ranking the considered strategy options, the strategies with the highest ranking matching agency needs can be selected.

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Consequence Analysis of Left-turn Driver's Scrambling Behavior to Traffic Efficiency at a Two-phase Signalized Intersection

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ABSTRACT

Urbanization and motorization have increased rapidly in China during last decades, along with China's economic boom. The increasing amount of motor vehicles brings frequent congestions, crashes, pollution and other negative impacts on the metropolis. Meanwhile, aberrant driving behaviors often happen among Chinese drivers and even enhances those negative impacts. Scrambling behavior is a common aberrant driving behavior, and always occurs at two-phase signalized intersection without left-turn signal, i.e. some left-turn drivers will "scramble" to pass through the intersection and not yield the way to the straight flow coming from the opposite direction, which has a higher priority, thus may increase the risk of crash and delay at the intersection. This paper focuses on the effect of this scrambling driving behavior on traffic efficiency. By analyzing the character of left-turn driver's behavior and calculating the delay on the conflict point, some findings are presented: (1) The stopping time of the straight flow caused by average scrambling vehicle on the conflict point, or the average time a scrambling vehicle uses to pass through the conflict point, is linear proportional to the number of scrambling vehicles; (2) The scrambling behaviors significantly increase the delay of traffic flow and decreases the efficiency of the whole intersection. A theoretical model to estimate the delay of the straight-going vehicle on the conflict point is proposed in this study. This paper can help to understand scrambling behavior and provide some basis to the practical operating process on signalized intersection in China.

1. INTRODUCTION

The number of motor vehicles is booming in current China, at the same time, under the influence of special driving culture, driving environment and other factors, aberrant driving behaviors also occur frequently (Shi et al., 2010). These

behaviors not only threaten the safety of drivers and other road users, but may also disrupt the normal traffic flow. Aberrant driving behaviors were classified into three categories in the beginning (Reason et al., 1990), including violation, which means “deliberate deviations from those practices believed necessary to maintain the safe operation of a potentially hazardous system”, errors, which means “the failure of planned actions to achieve their intended consequences”, and lapses, which means “failures of memory and attention”. Then this factor structure of aberrant driving behavior was extended by many studies (Blockey and Hartley, 1995; Kontogiannis et al., 2002; Li et al., 2004), and scrambling behavior achieves attention of Chinese researchers for causing many accidents. Scrambling behavior was defined as the behaviors of drivers, pedestrians, or cyclists that challenge for right of way in violation of traffic codes (Shi et al., 2011). This phenomenon often appears at intersection, roundabout and crosswalk, which contains traffic conflict points. According to research by Mao et al. (2009), scrambling behaviors of Chinese drivers accounted for 14.4% of road traffic accident.

Two-phase signalized intersection is a typical spot where scrambling behavior always take place. Without dedicated left-turn signal, the left-turn vehicle flow will conflict with the straight flow coming from the opposition direction. To pass through the intersection quickly, some left-turn drivers may “scramble” and not yield the way to the straight flow coming from the opposite direction, though the traffic law stipulates that the straight flow has the priority and other flows should not influence the movement of straight flow. Due to left-turn driver’s scrambling behavior, the risk of collision accident increases for the drivers that go straight may not have enough time to brake, and the straight flow will be delayed if it is disturbed by the left-turn vehicle with scrambling behavior, and hence may lower the traffic efficiency at the whole intersection.

Previous study relevant to scrambling behavior focused on its effect on traffic safety (Shi, et. al, 2011), and various traffic flow models of two-phase signalized intersection did not take the left-turn driver’s scrambling behavior in account but assumed the priority of straight flows. Thus, this paper try to investigate the effect of scrambling behavior on traffic efficiency, taking left-turn driver’s scrambling behavior at a two-phase signalized intersection as research object.

To better understand the left-turn driver’s scrambling behavior, the character of this behavior will be analyzed and explicated first. After that, we statistically calculate the delay of straight flow caused by scrambling behavior, then estimate the whole delay on the conflict point and finally propose a model to estimate the effect of left-turn driver’s scrambling behavior on efficiency of the intersection.

2. CHARACTER OF SCRAMBLING BEHAVIOR AT A TWO-PHASE SIGNALIZED INTERSECTION

Located in Haidian District of Beijing, Chengfu Road and Heqing Road intersection is a two-phase signalized intersection with no left-turn phase. In this intersection, the left-turn flow from the west entrance and the straight flow from the east entrance are the main conflict flows. As shown in Figure 1, left-turn drivers' scrambling behavior occurs frequently, especially during rush hours. Therefore, this intersection is chosen as study case.



Figure 1. Left-turn Driver's Scrambling Behavior during Rush Hour

If obeying the traffic law, the left-turn drivers should wait before the conflict point to yield the way to the straight flow until a suitable time gap appears. This suitable time gap is enough for left-turn vehicle to pass through the conflict point and not influence the movement of the straight-going vehicle after the gap. But in fact, when a left-turn vehicle passes through the conflict point within a suitable time gap, the vehicles running after it will scramble and follow preceding vehicle closely. So the straight flow will be blocked until another critical gap appears in left-turn flow.

The straight flow coming from the east entrance is much heavier than the left-turn flow coming from the west entrance, so, to prevent the left-turn vehicles from occupying the conflict point first, the signal of this intersection is revised to let the straight vehicles start first after the red time and then release the flow coming from the opposite entrance (west entrance). For there is few vehicles coming from the east entrance will turn left, this revise of signal influences the straight flow coming from the west entrance little.

3. STOPPING TIME OF STRAIGHT FLOW CAUSED BY SCRAMBLING BEHAVIOR

Without left-turn drivers' scrambling behavior, the straight flow coming from the opposite direction will pass through the intersection smoothly and the stopping time before the conflict point is zero. But situation will be changed if scrambling

behaviors exist, so it is necessary to record the number of scrambling vehicles and the time the scrambling vehicles need to pass through the conflict point, which is also the stopping time caused by scrambling vehicle platoon, to estimate the change.

By observing 84 platoons of scrambling vehicles with different scale, the stopping time of straight flow caused by average scrambling vehicle could be statistically calculated, and its relationship with the number of scrambling vehicles is found to significantly fit the linear regression. As illustrated in Figure 2, the stopping time caused by average scrambling vehicle increases linearly promotional to the number of scrambling vehicle.

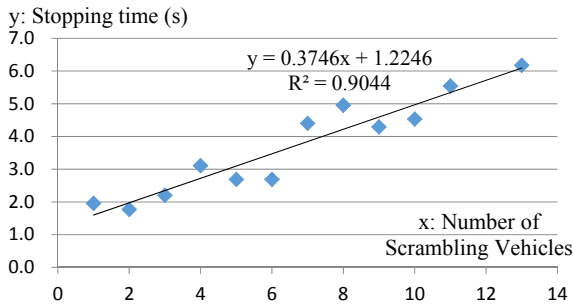


Figure 2. Stopping Time of Straight Flow Caused by Average Scrambling Vehicle

The more scrambling left-turn vehicles, the more average time a scrambling vehicle uses to pass through the conflict point and the average velocity of the scrambling vehicles will be lower at the intersection. That is because scrambling behavior makes the left-turn vehicles stop-and-go in platoon. Then, more pedestrians crossing signal lamp will zip through the lower-speed left-turn flow, and with the combined impacts from merging right-turn flow, the scrambling vehicles will be delayed, thus the straight-going vehicle need to be stopped before the conflict point for a longer time. Basing on the linear relationship, it is obvious that the total stopping time of straight flow caused by scrambling behavior has a quadratic relationship with the number of scrambling vehicles in a platoon.

Left-turn vehicles “scramble” to pass through the conflict point to save the waiting time before the conflict point, thus the straight-going vehicles have to wait instead and their delay increases. Without scrambling behaviors, the straight-going vehicles can pass through the intersection smoothly with a steady free flow velocity, while the scrambling vehicles pass through the intersection with a lower speed and bring a quite long stopping time to the straight flow. With above-mentioned linear relationship, the total delay of the straight flow caused by scrambling behavior will exceed the time that scrambling vehicles save on the conflict point. Therefore, the delay at the intersection increases and the efficiency of the whole intersection decreases.

4. DELAY OF STRAIGHT-GOING VEHICLES

The influence of scrambling behavior on the straight flow can be estimated from the delay of straight-going vehicles, which can be calculated by subtracting the time of each vehicle used to pass through intersection smoothly from the time with the influence of red time and waiting time caused by scrambling behavior. Figure 3 is the spatiotemporal diagram of straight-going vehicles, it can help to understand the effect of scrambling behavior on delay of straight-going vehicles, and the delay of the i^{th} vehicle behind the stop line can be calculated with the following equation:

$$d_i = d_1 + \sum_{k=1}^{i-1} h_k - \sum_{k=1}^{i-1} a_k \quad (1)$$

Where d_1 is the delay of the first straight-going vehicle behind the stop line of the entrance; h_i is the headway between the i^{th} and $(i+1)^{\text{th}}$ vehicle when passing through the stop line of the opposite direction; a_i is the headway between the i^{th} and $(i+1)^{\text{th}}$ vehicle when passing through the end point of the longest queue behind the stop line of the entrance.

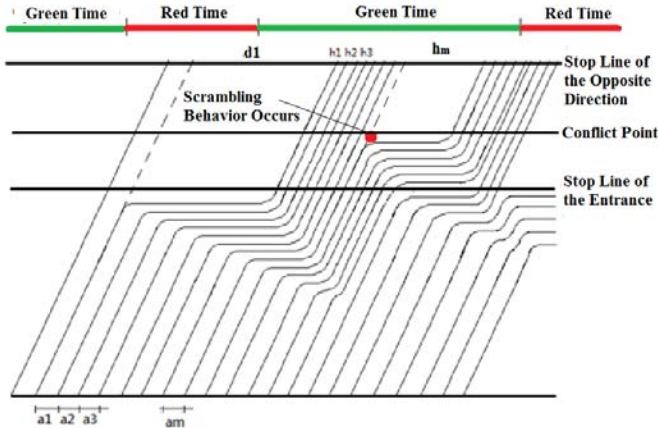


Figure 3. Spatiotemporal Diagram of Straight-going Vehicles

In the intersection, green time for Chengfu Road (east and west entrances) lasts 105 seconds. Using equation 1, delay of each straight-going vehicle within a signal cycle during a rush hour on weekday is calculated and illustrated in Figure 4: because 14 left-turn scrambling vehicles pass through the conflict point

between the 7th and 8th of straight-going vehicles behind the stop line of entrance, straight-going vehicles suffer a much longer delay from the 7th one.

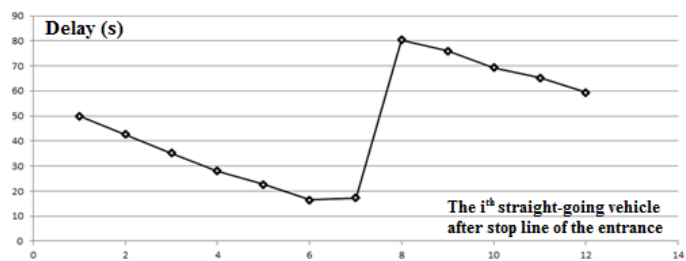


Figure 4. Delay of Straight-going Vehicles with Effect of Scrambling Behavior

Procedure of queue dissipation after the scrambling vehicles on the conflict point is the same as such procedure after red time on the stop line of the entrance. Thus, the stopping time of straight flow caused by the scrambling behavior can be regarded as another red time, and the duration of this “red time” has a quadratic relationship with the number of scrambling vehicle, which is always the queue length of left-turn vehicles before the conflict point.

Instead of calculating the delay within a signal cycle, we observe more straight-going vehicles within half an hour (5:30 p.m. ~6:00 p.m.) during rush time on a weekday, which is shown in Figure 5, and the average delay of the observed 203 vehicles is 60.42 s.

To investigate the effect of scrambling behavior, a left-turn signal is attempted to be installed, and the left-turn green time is set as 35s to ensure left-turn vehicles can pass through the intersection within a green time at most time. So the red time for the straight-going vehicles will increase by 35s correspondingly. The calculation indicates that, even though the green time for the straight-going vehicles decreases, the average delay is reduced to 34.99s, while the capacity of this entrance keeps the same and the delay of left-turn flow is also reduced.

By comparing the delay of straight-going vehicles without a left-turn signal and the delay when installing a left-turn signal (Figure 5 and Figure 6), we can conclude that, scrambling behavior significantly decreases the efficiency of traffic at two-phase signalized intersection, especially during rush hour. A left-turn signal can be installed to eliminate conflict point when the delay of straight-going vehicles is increased largely by scrambling behavior.

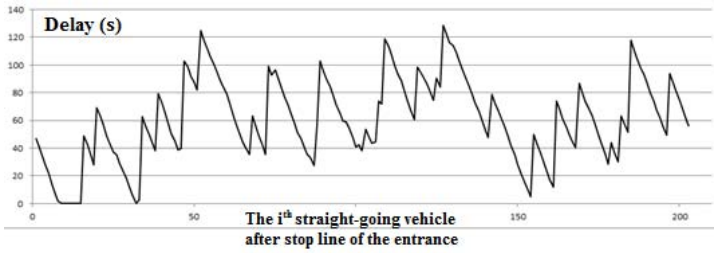


Figure 5. Delay of Straight-going Vehicles without a Left-turn Signal

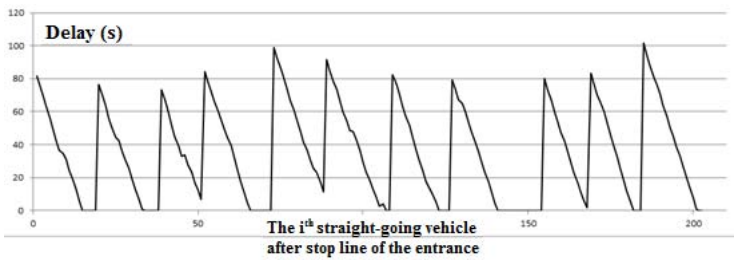


Figure 6. Delay of Straight-going Vehicles with a Left-turn Signal

5. A THEORETICAL LEFT-TURN CONFLICT DELAY MODEL

The above calculation of delay is based on statistical methods, however it cannot be generalized to apply to other intersections. Meanwhile, the efficiency of an intersection should consider the delay of all directions, not only the straight-going direction. Thus, a theoretical left-turn conflict delay model is tried to be proposed to provide some basis to the practical operating process on signalized intersection.

As mentioned in section 4, the stopping time of straight flow caused by the scrambling behavior can be regarded as another red time, if the start time t ($t=0$ when the signal turn green for that direction) and duration D of this “red time” are obtained, the delay of both the straight-going vehicles and left-turn vehicles can be calculated by established red-time delay model.

Considering the length of the first scrambling left-turn vehicle platoon is longest and this platoon influences the straight flow most, so we assume that scrambling behavior only occurs once within a green time for a direction. Basing on gap acceptance theory and queue theory, the average start time of the scrambling behavior can be estimated by equation 2 (Ma, et al., 2010), with the hypothesis that the headways of straight flow satisfy the Cowan’s M3 (Cowan, 1975) distribution:

$$\bar{t} = (\alpha e^{-\lambda(t_c - t_m)} - 1)(T e^{-\lambda_0 \alpha e^{-\lambda(t_c - t_m)T}} + \frac{e^{-\lambda_0 \alpha e^{-\lambda(t_c - t_m)T}} - 1}{\alpha \lambda_0 e^{-\lambda(t_c - t_m)}}) \quad (2)$$

Where $\lambda = \frac{\alpha q_1}{1 - t_m q_1}$, $\lambda_0 = \frac{\lambda(1 - e^{-\lambda t_c})}{1 - (1 + \lambda t_c)e^{-\lambda t_c}}$

α = proportion of free vehicles in straight flow;

q_1 = straight flow volume in veh/sec;

t_m = the minimum headway in straight flow;

t_c = critical gap time for left-turn vehicle to scramble;

T = green time for straight-going vehicles.

If the queue length L of left-turn vehicles before the conflict point is obtained, the duration D of this “red time” can be calculated from the quadratic relationship. Since:

$$L = q_2(R + \bar{t} + D) \quad (3)$$

and $D = (aL + b)L \quad (4)$

With equation 3 and 4, the average queue length L is not difficult to get:

$$L = \frac{1 - bq_2 + \sqrt{(b^2 - 4aR - 4a\bar{t})q_2^2 - 2bq_2 + 1}}{2aq_2} \quad (5)$$

Where α and β are parameters, q_2 is left-turn flow volume, and R is the

red time for the left-turn flow. After calculation of average start time $\bar{t}$ and average duration D of the scrambling behavior, the delay of straight-going vehicles and left-turn vehicles can be computed with established red-time delay model.

6. CONCLUSION

This study focuses on the effect of scrambling behavior on traffic efficiency, and chooses left-turn drivers' scrambling behavior at a two-phase signalized intersection as research object.

By analyzing the character of scrambling behavior and its effect on the delay of straight flow, it is concluded that: more scrambling vehicles lead to longer stopping time that each of them bring to the straight flow, and the total delay of the straight flow caused by scrambling behavior will exceed the time that scrambling vehicles save on the conflict point. Therefore, scrambling behavior significantly decreases the efficiency of traffic of the whole two-phase signalized intersection. A left-turn signal can be installed to eliminate conflict point when the delay of straight-going vehicles is increased largely by scrambling behavior.

A theoretical left-turn conflict delay model with the effect of scrambling behaviors is proposed, which can be applied to calculate delay in other intersection.

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An Optimization Method of Transportation Network Design Based on the Concept of Sustainable Development

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ABSTRACT

Traffic congestion, energy shortcoming and environment pollution are nowadays the common problems faced by many metropolises, which have severely restricted the further development of these cities; Therefore, the sustainable development of urban transportation system has aroused more and more attention. Based on this background, this paper attempts to investigate the transportation network design problem from the perspective of sustainable development, and focuses on the impacts of road broadening on total travel time, total fuel consumption and total vehicular emission at the network level. We integrate these three indicators into a sustainable development index, and then establish the corresponding bi-level optimization model. The upper level problem is to maximize the sustainable development index from the viewpoint of traffic managers, and the lower level problem is to depict travelers' route choice behavior based on user equilibrium (UE) theory. Finally, the genetic algorithm is used to solve the bi-level model. The numerical example shows that, the model and algorithm are feasible, and can be taken as a basis for the related research.

INTRODUCTION

The conventional Network Design Problem (NDP) can be broadly posed in two different forms. The first form is the Discrete Network Design Problem (DNDP) that determines the optimal locations of some new roads added to the current transportation network, while the second form is the Continuous Network Design Problem (CNDP) that determines the optimal capacity enhancements for some existing roads. Both forms of NDP attempt to minimize the total system costs under a budget constraint, while considering the route choice behavior of network users (Yang and Bell, 1998). This paper focuses on the impacts of road broadening on total travel time, total fuel consumption and total vehicular emission at the network level, thus it can be classified into the continuous version of the NDP.

Generally, a continuous network design problem (CNDP) can be formulated as a bi-level program. The upper level could be a multi-objective model or a model with objective function defined as the sum of total travel time and total generalized costs of fuel consumption and vehicular emission on the network under the constraint of total investment budget. The lower level problem is formulated as a certain traffic

assignment model, which can be either a static traffic assignment model or dynamic traffic assignment model. This paper integrates total travel time, total fuel consumption and total vehicular emission into a sustainable development index, and then establishes the corresponding bi-level optimization model.

As for the solution approaches to the CNDP, it is well known that the bi-level programming problem is one of the most difficult problems to solve in the optimization. Generally, such a bi-level model of CNDP is non-convex and non-differentiable and is quite difficult to solve with traditional optimization algorithms. Therefore, some heuristic methods such as genetic algorithm (GA) and simulated annealing (SA) are preferable to be used in solving the model for the globality, parallelism and robustness possessed by these algorithms.

In this paper, the CNDP has been studied using genetic algorithm (GA) on a simple network. The lower level problem is formulated as static user equilibrium traffic assignment model and Frank-Wolf method is used to solve it. The paper is organized as follows: Section 2 presents the bi-level programming model of CNDP based on the concept of sustainable development. Section 3 introduces the solution approach. Section 4 presents a numerical example. Section 5 concludes the paper.

BI-LEVEL PROGRAMMING MODEL

The CNDP based on the concept of sustainable development with fixed demand static user equilibrium flow constraint is formulated as:

$$\begin{aligned} \max \quad & I(\mathbf{x}, \mathbf{y}) = \exp\left(-\frac{Z(\mathbf{x}, \mathbf{y})}{10^m}\right) \\ Z(\mathbf{x}, \mathbf{y}) = & \sum_a [\alpha t_a(x_a(\mathbf{y}), y_a) x_a(\mathbf{y}) + \beta E_a(x_a(\mathbf{y}), y_a) x_a(\mathbf{y}) + \gamma F_a(x_a(\mathbf{y}), y_a) * \frac{l_a}{100} x_a(\mathbf{y})] \\ E_a(x_a(\mathbf{y}), y_a) = & 0.2038 t_a(x_a(\mathbf{y}), y_a) \exp(0.7962 \frac{l_a}{t_a(x_a(\mathbf{y}), y_a)}) \\ F_a(x_a(\mathbf{y}), y_a) = & 0.002 [\frac{l_a}{t_a(x_a(\mathbf{y}), y_a)}]^2 - 0.3465 \frac{l_a}{t_a(x_a(\mathbf{y}), y_a)} + 26.135 \\ \text{s.t.} \quad & 0 \leq y_a \leq u_a \quad \forall a \in A \\ & \sum_{a \in A} g_a(y_a) \leq B \end{aligned}$$

where $\mathbf{x}(\mathbf{y})$ is the equilibrium flow defined by the following fixed demand static user equilibrium problem:

$$\begin{aligned} \min \quad & z(\mathbf{x}) = \sum_a \int_0^{x_a} t_a(\omega, y_a) d\omega \\ \text{s.t.} \quad & \sum_k f_k^{rs} = q^{rs} \quad \forall r \in \mathbf{R}, s \in \mathbf{S} \\ & x_a = \sum_{rs} \sum_{k \in K^{rs}} \delta_{a,k}^{rs} f_k^{rs} \quad \forall r \in \mathbf{R}, s \in \mathbf{S}, a \in \mathbf{A}, k \in \mathbf{K}^{rs} \\ & f_k^{rs} \geq 0 \quad \forall r \in \mathbf{R}, s \in \mathbf{S}, k \in \mathbf{K}^{rs} \end{aligned}$$

The notations are defined as follows:

A	the set of links in the network
R	the set of origins
S	the set of destinations
K^{rs}	the set of paths between OD pair rs , $\forall r \in R, s \in S$
f	the vector of path flows, $f = [f_k^{rs}]$, $\forall r \in R, s \in S$
x	the vector of link flows, $x = [x_a]$, $\forall a \in A$
y	the vector of link capacity expansions, $y = [y_a]$, $\forall a \in A$
u	the vector of upper bound for link capacity expansions, $u = [u_a]$, $\forall a \in A$
B	the total investment budget
m	digit number of the function value $Z(x,y)$, and $10^{m-1} \leq Z(x,y) \leq 10^m$
t	the vector of link travel times, $t = [t_a(x_a, y_a)]$, $\forall a \in A$
E_a	the emission per vehicle on link a
F_a	100km fuel consumption per vehicle on link a
g	the vector of investment costs, $g = [g_a(y_a)]$, $\forall a \in A$
$\delta_{a,k}^{rs}$	the link-path incidence, $\delta_{a,k}^{rs}=1$ if link a is on the k th route connecting origin r and destination s , and $\delta_{a,k}^{rs}=0$ otherwise
α, β, γ	the monetary conversion factors

In this bi-level model, the upper level describes leader or policy problem and the lower-level model represents follower or user's behavioral problem. In the CNBP based on the concept of sustainable development, the upper-level problem is to determine an optimal decision for road broadening to make the sustainable development index maximum in the range of investments budget formulated by the government. The lower-level problem represents a user equilibrium assignment problem that describes user's path-choosing behavior, and its objective function is to minimize the user's travel cost.

SOLUTION APPROACH

Genetic algorithm (GA) is a global search heuristic technique used in computing to find true or approximate solutions to optimization problems. It uses techniques such as selection, crossover, and mutation which are inspired by evolutionary biology. Genetic algorithms are implemented as a computer simulation in which a population of chromosomes of candidate solutions to an optimization problem evolves toward better solutions. Solutions can be represented in binary or real-coded. The evolution usually starts from a population of randomly generated individuals and happens in generations. In each generation, the fitness value of every individual in the population is evaluated, multiple individuals are randomly selected from the current population (based on their fitness value) and modified (recombined and possibly randomly mutated) to form a new population. The new population is then used in the next iteration of the algorithm. Commonly, the algorithm terminates when either a maximum number of generations has been produced, or a satisfactory fitness level has been reached for the population.

A real-coded genetic algorithm with multiple crossovers was used in this paper. The algorithm is well described by Chang (2007). A brief summary of the algorithm is given below for completeness.

(1) Initialization. A search space should be first defined, and all the chromosomes will be operated and evaluated in this constrained space. Set the maximum number of generations as T , population size as N , crossover probability as P_c , mutation probability as P_m . Then Set $t=0$, and create the original population $[\text{Pop}(0)]$ consisting of N chromosomes, which can be denoted as $[\text{Pop}(0)]_1, [\text{Pop}(0)]_2, \dots, [\text{Pop}(0)]_i, \dots, [\text{Pop}(0)]_N$. It is noteworthy that these chromosomes are randomly generated from the feasible region.

(2) For each chromosome $[\text{Pop}(t)]_i$ ($i=1,2,\dots,N$) in the population $[\text{Pop}(t)]$, solve the lower level program by Frank-Wolfe algorithm and evaluate the fitness value with the sustainable development index, which is the objective function of the upper level program.

(3) If $t > T$, then stop and output the optimal chromosome; else continue.

(4) Reproduction. In this process chromosomes are copied according to their fitness values (i.e. the sustainable development index), and roulette wheel selection is adopted to reproduce the offspring chromosomes. The resulting population has the same size with the original one.

(5) Crossover. The N chromosomes are randomly divided into $N/2$ pairs. Assume that both $\mathbf{y}$ and $\mathbf{y}'$ are selected and c is a random number chosen from $[0, 1]$. If $c > p_c$, then the following crossover operations for $\mathbf{y}$ and $\mathbf{y}'$ are performed

if $I(\mathbf{y}) > I(\mathbf{y}')$ then $\bar{\mathbf{y}} = \mathbf{y} + \lambda(\mathbf{y} - \mathbf{y}')$, $\bar{\mathbf{y}}' = \mathbf{y}' + \lambda(\mathbf{y} - \mathbf{y}')$;

else, $\bar{\mathbf{y}} = \mathbf{y} + \lambda(\mathbf{y}' - \mathbf{y})$, $\bar{\mathbf{y}}' = \mathbf{y}' + \lambda(\mathbf{y}' - \mathbf{y})$

where $I(\mathbf{y})$ and $I(\mathbf{y}')$ are fitness values of chromosomes $\mathbf{y}$ and $\mathbf{y}'$, respectively, $\bar{\mathbf{y}}$ and $\bar{\mathbf{y}}'$ are the resulted children chromosomes, and $\lambda \in [0,1]$ is a random number determining the crossover grade of these two. If $c < p_c$, no crossover operation is performed.

(6) Mutation. Uniform mutation is adopted and only $p_m N$ random chromosomes in the population $[\text{Pop}(t)]$ are chosen to be mutated. The formula of mutation operation for a selected $\mathbf{y}$ is given by

$$\mathbf{y}_{\text{new}} = \mathbf{y} + \alpha \boldsymbol{\theta}$$

where α is a small positive constant and $\boldsymbol{\theta}$ is a random perturbation vector to produce small disturbances on $\mathbf{y}$. If the resulting chromosome is outside the feasible region, the original one is retained.

(7) Set $t := t+1$, then go back to step (2).

NUMERICAL EXAMPLE

To illustrate the model and algorithm proposed in this paper, we apply it to a simple network with five nodes, seven links, and two OD pairs, as given in Figure 1. Both the travel demands between OD pair (1, 3) and (5, 3) are 55 units. The link performance function is the standard BPR function below, and the link investment cost function is $g_a(y_a) = 1.5d_a * (y_a)^2$. All the link related parameters are provided in Table 1.

$$t_a = t_a^0 [1 + 0.15 (\frac{v_a}{C_a})^4], \quad a = 1, 2, \dots, 7$$

where t_a , t_a^0 , v_a , and C_a are the travel time, free-flow travel time, flow, and capacity of

link a, respectively.

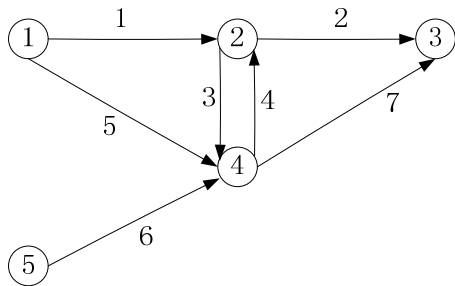


Figure 1. A Simple Network

Table 1. Link Related Parameters of the Example Network

Link	1	2	3	4	5	6	7
Free-flow travel time (t_a^0)	15	10	10	15	10	15	10
Capacity (C_a)	$40+y_1$	$45+y_2$	$40+y_3$	$35+y_4$	$40+y_5$	$35+y_6$	$40+y_7$
Link length(l_a)	25	30	25	30	30	25	25
Upper bound of y_a	5	10	10	5	10	5	10
Investment parameter(d_a)	2	2	1	1	3	2	2
Investment budget(B)	800						

Table 2. Results of the Example and the Resultant Link Flow Pattern

Link	1	2	3	4	5	6	7
Link expansion (y_a^*)	2.12	8.98	0.00	0.00	6.14	5.00	10.00
Link flow (x_a^*)	37.32	37.32	0.00	0.00	17.68	55.00	72.68
Link travel time (t_a^*)	16.34	10.33	10.00	15.00	10.03	23.04	16.82

Table 3. Corresponding Route Flow Pattern and Route Cost

OD	(1, 3)				(5, 3)	
Routes	1-2-3	1-2-4-3	1-4-3	1-4-2-3	5-4-3	5-4-2-3
Links on route	1,2	1,3,7	5,7	5,4,2	6,7	6,4,2
Route flow (f_{rs}^*)	37.32	0.00	17.68	0.00	55.00	0.00
Route cost (c_{rs}^*)	26.73	43.16	26.73	35.37	39.74	48.37

In this paper, Matlab GA Toolbox is applied to solve the bi-level programming model, with the GA parameters set as follows: $T=100$, $N=20$, $P_c=0.65$, $P_m=0.01$.

The final output of this example is $[y_1 \ y_2 \ y_3 \ y_4 \ y_5 \ y_6 \ y_7]=[2.12 \ 8.98 \ 0.00 \ 0.00 \ 6.14 \ 5.00 \ 10.00]$, and the corresponding link flow is $[x_1 \ x_2 \ x_3 \ x_4 \ x_5 \ x_6 \ x_7]=[37.32 \ 37.32 \ 0.00 \ 0.00 \ 17.68 \ 55.00 \ 72.68]$, as shown in table 2 and table 3, and the resultant

sustainable development index is 0.755, which is the maximum value of the objective function.

From table 2 and table 3 we can see that there are 4 routes between OD pair (1, 3) and 2 routes between OD pair (5, 3), and the total travel costs on all the used routes between the same OD pair are equal and less than the costs on all the unused routes, which is entirely consistent with the Wardrop's UE principle.

CONCLUSION

In this paper, the continuous network design problem (CNDP) based on the concept of sustainable development has been studied. The CNDP is formulated as a bi-level programming problem: the upper level model is to find the optimal scheme of network expansion from the perspective of sustainable development and the lower level is to depict the users' travel behavior. Moreover, this paper integrates total travel time, total fuel consumption and total vehicular emissions into a sustainable development index, which is the objective function of upper level model. Besides, genetic algorithm (GA) is adopted to solve the upper level model, while Frank-Wolfe algorithm is used to solve the lower level traffic assignment problem. The numerical example shows that, the model and algorithm are feasible, and can be taken as a basis for the related research.

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The Study on Right-Turn Vehicle Capacity under Mixed Traffic Flow Conditions

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ABSTRACT

Traffic conflicts exist among mixed traffic conditions. Considering influences of pedestrians, bicycles and mixed traffic flows, this paper first summarized the right-turn capacity calculation methods based on conflict analysis by applying the gap theory. Furthermore, the key of this paper is the right-turn capacity calculation formula based on statistical analysis, i.e., the vehicle capacity is presented as a curve with the capacity as the vertical axis and the pedestrian or bicycle flow as the horizontal axis, which is fitted using traffic survey data of six typical intersections in Beijing. Comparing above two methods upon influences from pedestrians, bicycles and mixed traffic flows, the characteristics of right-turn capacity in different situations could be acquired. By comparing the queue length obtained from inputting right-turn capacity calculation data from this paper with that from existing methods, the results of this paper were closer to the practical situation and more accurate, because the calculation formula of this paper not only applied the gap theory, but also adequately took the influence of pedestrians, bicycles and mixed traffic flows into account.

Keywords: mixed traffic flow; capacity; right-turn conflict; conflict analysis; statistical analysis

INTRODUCTION

There exist acute traffic conflicts of right-turn vehicles, bicycles and pedestrians in modern urban intersections since the diverse traffic flows cannot be separated easily by signal control methods or physical isolation measures. The interferences of bicycles and pedestrians extremely affect the right-turn capacity of vehicles. Therefore, it has been hot research issues of traffic engineering field to study how

to enhance the right-turn capacity in signal control intersections, improve travelling sequence in intersections and cut down inferences between vehicles and bicycles.

Bang and Carlsson (1994) presented bicycles and pedestrians both have influences on capacity of intersections which were relevant. Zhao (2004) built up the acceptable delay model of pedestrians and the acceptable gap model of bicycles related to vehicles in signal control intersections, furthermore, came up with the description theory and the quantitative mathematical model which could contribute to the microscopic simulation of signal control intersections under mixed traffic flows. Ren (2007) described some simple contents about the characteristics of bicycle flows, neglecting the conflict problem between vehicles and bicycles. Guo (2009) studied the traffic characteristics of bicycles within intersections and analyzed the conflicts between turning vehicles and bicycles from two phases, applying different coefficients to revise the vehicle capacity under influence of bicycles. Chen (2010) put forward the calculation model of vehicle capacity under mixed traffic conditions in signal control intersections based on the gap theory by regarding bicycles and pedestrians as the interference facets.

This paper attempts to apply the gap theory and the statistical analysis method to study the calculation methods of right-turn vehicle capacity under mixed traffic conditions.

THE RIGHT-TURN CAPACITY CALCULATION METHODS BASED ON CONFLICT ANALYSIS BY APPLYING THE GAP THEORY

Bicycles and pedestrians affect turning vehicles simultaneously. However, the affection of bicycles and pedestrians is relevant that it cannot be simply stacked up, because bicycles may momentarily hinder vehicle flows which indirectly avoid the conflicts between vehicles and pedestrians.

This paper used capacity calculation method based on gap theory which focused on the conflict area, took lanes or lane groups as the analysis object, and separately calculated vehicle capacities of different directions under influences of relevant pedestrians, bicycles and vehicles in conflict area. On the basis, according to the idea of “Barrel Theory”, this paper could find out the capacity bottleneck of different traffic flows by comparison, achieve traffic flow capacity, and make combination of influences from pedestrians, bicycles and vehicles to capacity.

Such as right-turn vehicles, when green light comes, bicycles firstly start to occupy the conflict area until intensive travelling ends. During the interval when bicycles stop the vehicle flows, pedestrians and vehicles don't have conflict momentarily. Then vehicles begin to move through the conflict point between pedestrians and vehicles.

Capacity under the mixed traffic flow conditions:

$$Cap_R = \min(Cap_{BR}, Cap_{PR}) \quad (1)$$

Among it, Cap_{BR} and Cap_{PR} separately present right-turn vehicle flow capacity influenced by bicycles and pedestrians.

Right-turn capacity influenced by bicycles

During a signal cycle, right-turn vehicle capacity influenced by bicycles:

$$Cap_{BR} = 3600(C_{none} + C_{wt} + C_{tr})/T_c \quad (2)$$

Among it, T_c --length of signal cycle; C_{none} --capacity at interval when vehicles turn right without slowing down(no bicycles influence); C_{wt} --capacity at interval when vehicles are influenced by bicycles waiting to pass through; C_{tr} --capacity at interval when vehicles slow down to pass through;

Right-turn capacity influenced by pedestrians

During a signal cycle, right-turn vehicle capacity influenced by pedestrians:

$$Cap_{PR} = 3600N_p/T_c \quad (3)$$

Among it: N_p --number of vehicles permitted passing through;

Right-turn vehicle capacity curve fitted by the gap theory

Based on capacity calculation methods above, this paper used traffic flow data in various situations to fit the capacity curve under influences of pedestrians, bicycles and mixed traffic flow conditions with the capacity as the vertical axis and the pedestrian or bicycle flow as the horizontal axis. There are totally 50 groups of successive data to fit the curves.

The fitted formula of capacity influenced by bicycles:

$$C_{br} = 2147.3 * e^{-0.0008V_b} \quad (4)$$

$$R^2 = 0.9874$$

Among it: C_{br} --right-turn vehicle capacity influenced by bicycles; V_b --number of bicycles; R^2 --fitted degree;

The fitted formula of capacity influenced by pedestrians:

$$C_{pr} = 1691.8 * e^{-0.0011V_p} \quad (5)$$

$$R^2 = 0.9988$$

Among it: C_{pr} --right-turn vehicle capacity influenced by pedestrians; V_p --number of pedestrians; R^2 --fitted degree;

It is not difficult to find two fitted degree are all approaching 1 which means the fitted formula is reliable, the theory is superior, and results are trustworthy.

THE RIGHT-TURN CAPACITY CALCULATION FORMULA BASED ON STATISTICAL ANALYSIS

The right-turn capacity calculation formula based on statistical analysis is realized by analyzing traffic survey data and data fitting. This paper selected typical intersections in Beijing to carry out traffic survey, especially collecting data of the area where right-turn vehicles conflict with pedestrians and bicycles, and achieved right-turn vehicle capacity under the mixed traffic flow conditions by acquiring time headway data.

Traffic survey

The elemental data needed contain time headway t_h , number of vehicles passing through, the difference between start time and end time Δt , number of pedestrians in different directions, and number of bicycles in different directions.

There are many non-signal control intersections in Beijing, and it is hard to get the saturated time headway. In order to get the saturated right-turn traffic flows and the unsaturated right-turn traffic flows, it is necessary to carry out traffic survey in right-turn signal control intersections.

It is easy to find that vehicles in queue usually reach their saturated time headway after the fifth vehicle passing through the stopping line in right-turn signal control intersections. It is obvious that starting to count right-turn vehicles after the fifth vehicle has leaving can get the saturated time headway data.

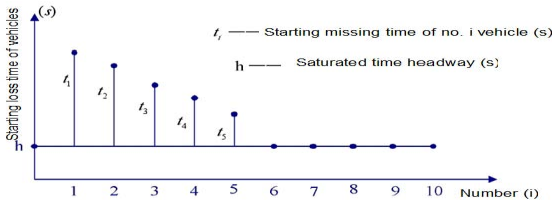


Figure 1. The Loss Time of Vehicles at Intersections

Data analysis

According to traffic survey results, there is no vehicle when time headway is from 0 to 1.5s or bigger than 7s. Time headway of most vehicles is from 1.5 to 7s.

(1) right-turn capacity influenced by bicycles

The fitted formula of capacity influenced by bicycles:

$$C_{br} = 1248.7 * e^{-0.0007V_b} \quad (6)$$

$$R^2 = 0.2923$$

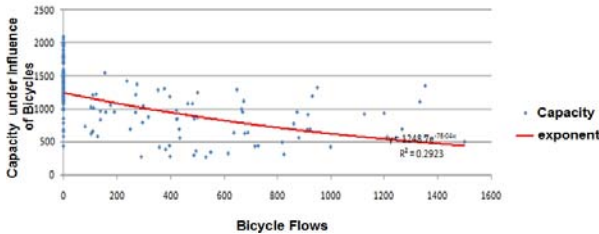


Figure 2. Right-Turn Capacity Influenced by Bicycles

(2) right-turn capacity influenced by pedestrians

The fitted formula of capacity influenced by pedestrians:

$$C_{pr} = 1242.3 * e^{-0.0002I_p} \tag{7}$$
$$R^2 = 0.205$$

The fitted curve above has separately shown tendency of capacity influenced by bicycles and pedestrians at some degree, and it is understandable that R^2 is relatively low for the data size is rather small.

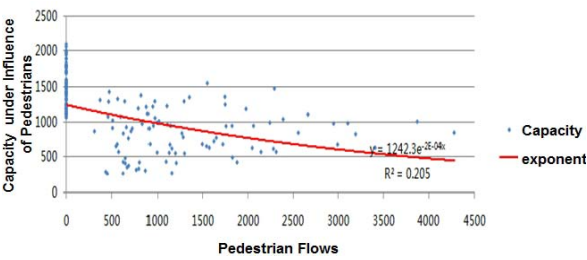


Figure 3. Right-Turn Capacity Influenced by Pedestrians

(3) right-turn capacity under mixed traffic flow conditions

In Figure 4, tendency of right-turn capacity under mixed traffic flow conditions can be acquired. When pedestrian and bicycle flows are 0, right-turn vehicle capacity is about 1200. When the quantities of pedestrians and bicycles increase, right-turn vehicle capacity declines as a whole with less regular because the quantities of pedestrians and bicycles are rather small which means pedestrians and bicycles pass through the street irregularly without group phenomenon and certain gap for vehicles to cross. When the quantities of pedestrians and bicycles are big, it is obvious that right-turn vehicle capacity declines in a smooth tendency with apparent regular as a whole. On this basis, the tendency of right-turn vehicle capacity under mixed traffic flow conditions can be judged.

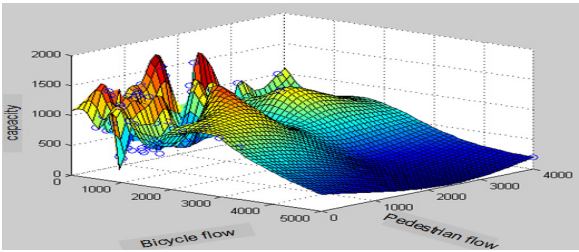


Figure 4. Tendency of Right-Turn Capacity under Mixed Traffic Conditions

Through separating right-turn capacity scatter gram from tendency diagram of right-turn capacity under mixed traffic flow conditions and wiping off some wrong points artificially, the fitted tendency diagram of right-turn capacity under mixed traffic flow conditions can be drawn as Figure 6 shows.

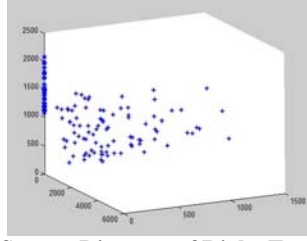


Figure 5. Scatter Diagram of Right-Turn Capacity

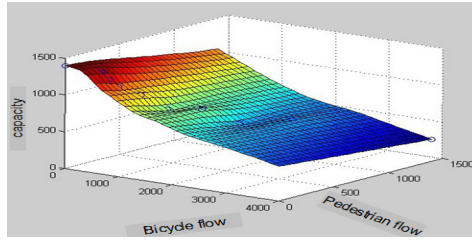


Figure 6. Fitted Tendency Diagram of Right-Turn Capacity under Mixed Traffic Flow Conditions

At the same time, the fitted formula of right-turn vehicle capacity under mixed traffic flow conditions can be acquired below:

$$Cap_R = -n_1 \times C_p - n_2 \times C_b + N \quad (8)$$

Among it: n_1 --pedestrians reduction coefficients (equal to 0.358, fitted by Matlab); n_2 --bicycles reduction coefficients(equal to 0.491, fitted by Matlab); C_p --quantity of pedestrians; C_b --quantity of bicycles; N --fitted constant(usually equal to 1406.3);

When C_p and C_b are all bigger than 1600, Cap_R approaches 0.

COMPARISON ANALYSIS

Comparison between the gap theory and statistical analysis method

By comparison between gap theory and statistical analysis method in bicycle influence and pedestrian influence, as Figure 7 and Figure 8 shows, the general tendencies are accordant. However, the result of gap theory method under bicycle influence is above that of statistical analysis method and pedestrian influence is on the contrary which means pedestrian influence is more complicated with influence of more factors decided by randomness of pedestrians. When the pedestrian flows are small, only several factors function, once they surpass 400, the influence increases heavily. Meanwhile, the influence of bicycles is stable, considering the speed of bicycles is bigger which leaves more time for vehicles to cross the gap.

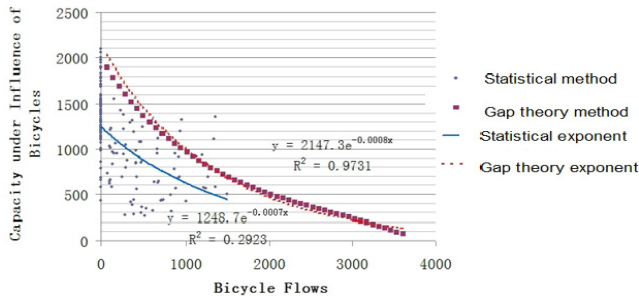


Figure 7. Comparison of Capacity under Influence of Bicycles

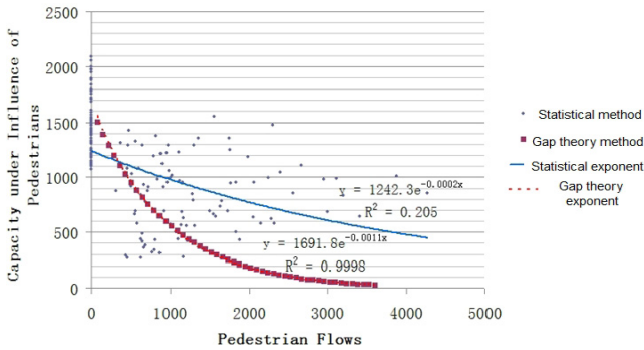


Figure 8. Comparison of Capacity under Influence of Pedestrians

Test of right-turn capacity of vehicles under the mixed traffic flow conditions by calculating queue length

This paper used queue length to compare right-turn capacity calculation data from this paper with that from existing methods because it is a critical index for non-signal control intersections and right-turn control is usually non-signal.

The formula of 95% queue length:

$$Q_{95} = 900T \left[\frac{v_x}{c_{m,x}} - 1 + \sqrt{\left(\frac{v_x}{c_{m,x}} - 1 \right)^2 + \frac{(3600/c_{m,x})(x/c_{m,x})}{150T}} \right] (c_{m,x}/3600) \quad (9)$$

Among it: v_x --traffic flow rate in x direction (pcu/h); $c_{m,x}$ --capacity in x direction (pcu/h); T --length of analysis interval (0.25);

Supposing traffic flow rate and analysis interval are same, the queue length calculated by common method through time headway and statistical analysis method is shown below:

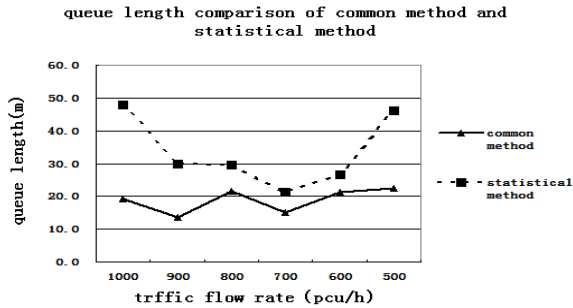


Figure 9. Queue Length Comparison of Common and Statistical Method

It is easy to find that the queue length calculated through statistical analysis method is obviously larger than common method which means queue length calculated by common method can only seemly reflect the status of right-turn vehicles without accurate result. On the other hand, the statistical method is on the basis of gap theory, the regular of mutual influence of pedestrians, bicycles and vehicles, and the fitted result by real traffic survey data. All in all, the statistical method is more accurate than the common method for queue length calculated by statistical method is bigger than common method by considering more influencing factor such as the influences of pedestrians and bicycles. In fact, only using the time headway to calculate capacity without considering other influencing factor is not accurate, and the statistical method used to calculate right-turn capacity under the mixed traffic flow conditions in this paper is more reasonable.

CONCLUSIONS

Once and for all, this paper put forward and realized the calculation methods of right-turn capacity under mixed traffic conditions by gap theory and statistical analysis method, and the main achievements can be concluded below:

(1) With the gap theory, this paper used traffic flow data in various situations to fit the right-turn vehicle capacity curve which means a lot to right-turn vehicle capacity calculation in non-signal control intersections.

(2) With traffic survey data of six typical intersections in Beijing, this paper acquired right-turn vehicle capacity by fitting the capacity curve based on statistical analysis, containing right-turn vehicle capacity influenced by pedestrians, bicycles or the mixed traffic flow conditions.

(3) Comparing the right-turn vehicle capacity calculated by gap theory method and statistical analysis method under bicycle influence and pedestrian influence, this paper concluded the differences between bicycle influence and pedestrian influence.

(4) Comparing the right-turn vehicle capacity calculated by the common method and the statistical method with queue length as the evaluation index, this paper summarized that the statistical method proposed is superior.

ACKNOWLEDGMENTS

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Discussion on the Promotion of Drop-and-pull Transport Organization Mode of Shandong Province Road Logistics Enterprises

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ABSTRACT

Drop-and-pull transport refers to a tractor, according to a predetermined operation plan, dropping a trailer in a cargo loading area and unloading. This paper introduces the significance of the development of drop-and-pull transport and the achievements of the drop-and-pull transport pilot project of Shandong province. There are problems with lag in dedicated infrastructure construction, regulations, policies, and standardized construction of models, as well as a lack of leading enterprises and cooperation among enterprises. By adjusting laws and regulations, strengthening the standardization of vehicle types, and constructing a freight market network, leading enterprises can solve the problems in the pilot project and promote the development of Shandong's drop-and-pull transport.

Key Words: Road freight transport; network transport; evolution; revelation

INTRODUCTION

In developed countries, drop-and-hook transport as an advanced transportation organization has been greatly developed. It is estimated that with drop-and-hook transport, vehicle transportation efficiency can be improved more than 30 percent, costs can be reduced by 30 percent, and vehicle fuel consumption can be reduced by 20–30 percent. (Qu 2008). Problems such as backwards transport organization, low transport efficiency, poor vehicle technology, high energy consumption, high logistics costs, and low quality of the overall operation exist in China's road transport logistics. The development of drop-and-pull transport "is the key to solving these problems. Therefore, road transport logistics should attach great importance to drop-and-pull transport and should seriously discuss the technological means, organizational conditions, and policy environment of drop-and-pull transport to promote its development.

IMPORTANCE OF DEVELOPING DROP-AND-PULL TRANSPORT

Drop-and-pull transport refers to a tractor, according to a predetermined operation plan, dropping a trailer at a cargo loading and unloading point and pulling other trailers to continue to run the transport organization (Xue 2011). Drop-and-pull transport includes auto/train transport and loading and unloading technology. It is an intensive,

highly efficient mode of transport, and its organizations are large-scale and networked and use information technology and standardization. Significant advantages are improved transport efficiency, reduced logistics costs, energy savings, and other aspects. Specifically, the following are of economic and social significance for carrying out drop-and-pull transport:

- Drop-and-pull transport can increase the pure running time of a tractor to dramatically improve the productivity of vehicle transport.
- Drop-and-pull transport can reasonably coordinate the time of transport and loading and unloading, increase the speed of operation of the logistics, and comprehensively improve the efficiency of logistics.

In drop-and-pull transport, the number of tractors can be reduced, so logistics companies can greatly reduce purchase and operating costs of tractors and can reduce vehicle occupancy of roads, as well as reduce road traffic pressure, energy consumption, and vehicle emissions pollution. As the number of tractors is reduced, companies can reduce the number of employed drivers, thereby reducing labor consumption and related expenses to improve overall labor productivity.

Drop-and-pull transport can promote multimodal transport in road, railway, and waterway transport by combining automobile drop-and-pull transport such as railway piggyback transport and roll on/roll-off waterway transportation (Wang 2011), giving full play to the various modes of multimodal transport, and can improve handling efficiency and capacity utilization.

Drop-and-pull transport should be developed to a higher degree of business organization. It can contribute to the construction and development of a logistics center and a cargo station logistics node and promote road transport and logistics network management, thus urging road transport and logistics companies toward intensive, large-scale development and enhancing their level of logistics services.

PERFORMANCE OF DROP-AND-PULL TRANSPORT OF SHANDONG PROVINCE

At the end of 2009, the Ministry of Transport, the National Development and Reform Commission, the Ministry of Public Security, the Customs Department, and the Insurance Regulatory Commission issued a Notice on the promotion of drop-and-pull transport development and identified a drop-and-pull transport pilot project. The Ministry of Transport and the National Development and Reform researched and developed a pilot implementation of the program.

Shandong province is one of the 10 provinces to pilot drop-and-pull transport. As a large province, it has had significant development in recent years (Table 1). In 2011, the total mileage on the province's highways was 233,000 km, including 4,350 km of highway. Provincial highway freight volume reached 2.79 billion tons, and freight turnover reached 662.44 billion tons. There were 815,000 road freight business operators, 1.266 million people were employed, goods vehicle operated were 1.2 million, and there were 46,800 tractors and 46,900 trailers in the province, the top in the nation.

Table 1. Comparison of Freight Transport in Shandong and the Nation

Year	National freight transport (million tons)	Freight transport, Shandong (million tons)	Percent of national total
2006	20,370.59	1,675.11	8.22
2007	2275821	1,985.07	8.72
2008	25,859.37	2,474.89	9.57
2009	28,252.22	2,844.63	10.06

Development plans were formulated and support efforts for policy support and funding were increased. To improve the drop-and-pull transport pilot in the first pilot, policies and measures were implemented to establish the road cargo drop-and-pull transport development plan of Shandong Province and ensure that the drop-and-pull transport had scientific and orderly development. Shandong province also increased its policy support and funding efforts to support the project. The Ministry of Transportation of Shandong Province invested 12 million Yuan as pilot funds and carried out pilot work in Jinan, Zibo, Yantai, and Dezhou City, including seven companies and six station pilots in two batches. After more than two years of the pilot, there were 360 tractors and 950 trailers in the pilot enterprises. Drop-and-pull transport was successfully achieved 36,000 times, which included 19,000 roll-on/roll-off drop-and-pull transports.

A successful organizational model of drop-and-pull transport was initially explored. To promote the drop-and-pull transport pilot, Shandong province adhered to scientific planning and reasonable adjustments to the road freight organizational structure and actively explored the methods of multimodal transport of other regions. From the linkage mechanism in production, processing, distribution, shipping port enterprises, and logistics, the drop-and-pull enterprises initially established an organization mode to precede successful drop-and-pull transport between land and sea and on land routes. From the beginning of 2008 to the end of 2009, land and sea drop-and-pull transport was successfully realized for more than 11,000 vehicles in the Yantai-to-Dalian route in the Persian Gulf, which obtained better efficiency. With the gradually-expanded influence of drop-and-pull transport, many logistics enterprises, such as Yantai Beer, Runda logistics, and Kongxing logistics, participated in land and sea drop-and-pull transport. At present, drop-and-pull transport vehicles have a stable operation in the Yantai-to-Dalian route, and more than 400 drop-and-pull transports have begun to take shape.

An information platform was initially established. The transport logistics public information platform established by the Shandong Province Department of Transportation released information regarding enterprise authentication services. Preparing for drop-and-pull land and sea transport in the Persian Gulf, the transport department in Yantai city connected the Beiming Logistics Co., Ltd. and the Salvage Bureau of Yantai to sign a framework cooperation agreement to build a common information platform. This effectively connected information about road freight transport, ship capacity, ticketing of the shipping company, and so on, and formed a

regional logistics public information platform that provided comprehensive, high quality, and efficient integrated logistics services to the parties of the transaction (Xu 2011). At the same time, it strengthened the construction of the freight business integrity certification system and ensured the credibility of the participating enterprises that relied on the public information platform. Currently, the information platform includes manufacturers and logistics companies and gradually shares information to the Yangtze River Delta and Pearl River Delta regions, expanding the range of radiation and enhancing development.

PROBLEMS IN THE DEVELOPMENT OF DROP-AND-PULL TRANSPORT

Lagging development of regulations and policies. Currently, trailers and tractors must pay compulsory insurance, implement mandatory retirement, and limit double trailers and full trailers on the road, which is not conducive to lower the cost of the use of trailers and restricts the purchase of trailers, to some extent hindering the development of drop-and-pull transport.

Lagging development of standardized construction of models. Achieving standardization and serialization among different manufacturers and different models of vehicles, to allow free combination and convenient circulation between different tractor and trailers, is not only the basic premise for the development of drop-and-pull transport, but also a guarantee for technology. Model standardization lagged for a long time, and the lack of standards between the tractors and trailers resulted in the phenomenon of “could not hang on or could not drag and could not drop” (Xiao 2011), which constrained drop-and-pull transport development.

Lagging development of dedicated infrastructure construction. Freight stations are the important infrastructure of drop-and-pull transport. When drop-and-pull transport develops, there is a need in the network for support. While stations are the most basic nodes of the network, their main function is to supply organization, distribution, transit, and vehicle management. Road transport logistics companies should develop large and dense networks in order to optimize the transport organization to achieve the best match of operation scheduling of vehicles and goods circulation and to reach higher mileage utilization. This is the most effective measure to reduce transportation costs and improve the companies’ core competitiveness. Only a few stations are suitable for drop-and-pull transport in Shandong province.

Lack of leading enterprises. Leading enterprises have a very prominent position and role in the transport industry. They have a strong advantage in capital, technology, equipment, information technology, business networks, quality, safety, human resources, and social reputation. They provide customers with high quality, efficient service and, as representatives of the industry’s advanced productive forces, they lead other companies and the industry in the direction of development. They rely on their own competitive advantage and master a large number of sources, attracting other companies to cooperate. They play the role of integrating social vehicles and optimizing resource allocation. At present, Shandong province road logistics enterprises are small-scale, in large quantities, and weak in strength. They have the same management structure and their business scope is similar; competition is weak in the market (Table 2). Industry concentration is low, resulting in a serious homogeneous

competition at low levels, and business behavior is not standardized; phenomena such as vicious competitions often occur. The road freight business model is single; a majority of enterprises offer only transportation, warehousing, freight agent services, and other individual services, and only a few provide logistics services and value-added services.

Table 2. Road Freight Transport Enterprises in Shandong Province

Number of enterprises	2007	2008
Number of freight transport enterprises	624,376	812,982
Number of ordinary freight transport enterprises	606,256	350,136
Number of dedicated freight transport enterprises	1,125	1,200
Number of container shipping enterprise	595	669
Number of transportation-related enterprises	27,880	5031
Number of logistics service	832	1,022

The entire industry lacks large enterprises, which greatly impacts the market. There are some larger companies (with hundreds of cars or thousands of vehicles) in the market that include other companies that contract with them. With the poor organization of the enterprise, there is a lack of initiative to open up market strategies and measures; the enterprises have weak organization and production management. This mode of operation is not conducive to improving the efficiency of vehicle use, making technological progress, or ensuring transport safety, and it is more difficult to adapt to the development needs of the modern logistics industry.

Lack of cooperation between enterprises. There are no good facilities in the freight network for round-trip cargo, which is the basis for drop-and-pull transport's smooth development. If there is no mutual inter-regional and inter-enterprise cooperation, both technical and economic advantages and benefits of the drop-and-pull transport are difficult. Drop-and-pull transport in Shandong province is faced with unbalanced development between the eastern and western regions, and there is a lack of cooperation and an uncoordinated situation between regions. The coastal economic development of Shandong province is rapid, and there are large volumes that are suitable for drop-and-pull transport; the central and western regions have relatively weak economic development with low drop-and-pull transport of cargo. Imbalances in regional development affect the concentration of goods and the resource allocation of transport capacity, especially as it relates to the lack of a stable two-way supply, which will inevitably affect the process of Shandong province road drop-and-pull transport development. In addition, although the proportion of the amount of freight exchanges of China's inter-provincial highway is small (less than 2%), inter-provincial freight miscommunication is still one of the factors affecting road drop-and-pull transport development. There is no appropriate drop-and-pull transport mechanism of cooperation between Shandong province and the neighboring provinces, which can easily lead to increased transportation costs and operational coordination issues of drop-and-pull transport vehicle in the transport channel.

STRATEGIES FOR DROP-AND-PULL TRANSPORT DEVELOPMENT

Adjusting the rules and regulations is a priority. It is proposed to revise and improve the existing legal norms and technical standards system and to effectively distinguish between trailers and tractors, as well as the management difference between them, establishing a traffic safety management and regulation system that takes the tractor as the main body. Close attention should be paid to modify the Road Traffic Safety Law and Regulations for the Implementation of Road Traffic Safety Law. With reference to foreign legislative practice, “vehicle drawn by tractor” (trailer) should be added to the provisions of regulatory principles, including vehicle insurance, inspection, registration, licensing, and so on. The legal restrictions on double trailers and full trailers should be canceled at the appropriate time. Security technology requirements should be prescribed through mandatory technical standards to provide the appropriate vehicle traffic requirements according to different technical conditions of the road. At present, highway conditions have reached the international advanced level, and vehicle manufacturing technology and capacity can adapt to the requirements of the development of car trains. Traffic insurance premiums are proposed and should be charged with reference to the Shenzhen model, such that drop-and-pull transport vehicles have unified traffic compulsory insurance on tractors. Trailer should not require insurance; the liability of the trailer should be part of the tractor insurance, and insurance payments related to traffic accidents should be the responsibility of the tractor. Related to tractor/trailer combinations, detaching and attaching a train car on the road can increase accident risk, so tractor traffic compulsory insurance rates should be applied.

Strengthen standardization of vehicle type. At present, the Ministry of Transport has developed road freight transport models restructuring objectives and has begun to examine vehicles in the road freight transport market in terms of safety, energy conservation, environmental protection, and axle load quality, and some alternative models have been announced. Shandong province, on the basis of comprehensive assessment of adequate market research and scientific conclusions, has recommended several road drop-and-pull transport standard models from which businesses can choose. The general requirements of the standard models can be maintained over a longer period, which is conducive to stability and can facilitate the implementation of the enterprise. Examples include encouraging road logistics enterprises to continuously adjust and optimize the transport structure, equipment, and structures; encouraging the development of vans and other specialized vehicles for transporting dangerous goods, refrigerated goods, and other special items; equipping small, fast vehicles for urban distribution; and actively promoting long haul transport vehicles towards the direction of large-scale containerization, offering professional, fast service, energy savings, and environmental protection.

Strengthen network construction of road stations. The construction of road transport stations in the province should be speeded up, and the relationship of public welfare and operating to achieve the organic combination of government support and the market mechanism should be encouraged. The principles for selecting a station site

should be established; it is necessary to consider the connections among the highway network, other modes of transport, the urban transport network, and the convenience of shippers. Stations should rely on the transport market and passenger and cargo distribution centers in order to avoid the “market without business” phenomenon to improve investment returns.

Support leading enterprises to lead the industry’s direction. Policy guidance should be strengthened and enterprises should be encouraged to develop an innovative operating model to improve their transport organization structure and to focus on cultivating leading backbone enterprises. Leading enterprises should promote the industry, management levels, and technical levels and comprehensively improve the process. Included in the pilot drop-and-pull transport enterprises in Shandong province were financial support, increased capacity, and approval of efforts to create good conditions for development. Communication and coordination should be improved to encourage the introduction of beneficial drop-and-pull transport development tax incentives.

Partnerships and collaboration between transport enterprises should be encouraged. Transport enterprises among large manufacturing companies, commercial enterprises, professional commodity markets, and freight stations can integrate capacity and supply resources and improve the degree of intensive, large-scale networks and organization. Drop-and-pull transport coordination and cooperation between various cities should be strengthened, and unified qualification standards and unified technical standards should be developed and local barriers should be broken down to enhance the competitiveness of the province's drop-and-pull transport and create a standardized, orderly, and healthy development of the regional market environment.

When road drop-and-pull transport is carried out in the inter-provinces, local protectionism and policy barriers in the development of transport integration system should be eliminated, and the communication mechanisms between provinces should be strengthened. Technical standards should be strengthened, and road transport enterprises should be encouraged and supported to meet the conditions to set up business outlets (branches) in different places. When conditions are ripe, the large national transport and logistics enterprises can be matched to cooperate with transport and logistics enterprises of the province.

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Structural Integration and Optimization: New Models of Metropolitan Station Distribution Planning - Case Study of Beijing

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ABSTRACT

Traffic congestion has become a vital issue of improvement the quality of living in largest cities. Transit station distribution has a great impact on traffic flows in a city. This paper takes Beijing as a case to analyze the causes of traffic congestion in terms of station distribution, transportation network and traffic flow. As a conclusion, it proposes a new mode of metropolitan transport network planning by integrating multiple transportation modes and optimizing regional distribution of stations. It plans to create three integrated transport hubs which integrate urban railways stations, long-distance bus stations with inner city bus stations. The hubs are evenly distributed on the east, north and south Sixth Ring of Beijing. One of advantages of the new mode is that suburb-to-center traffic flow could be reduced, and thus traffic congestion would be relieved. This paper also introduces a universal pattern of station distribution in big cities.

Keywords: station distribution, new mode, ideal model, Beijing

1. INTRODUCTION

Traffic congestion, which results in longer commute, air pollution, the deterioration of the health, as well as economic loss, has become a severe problem sustainable development China's large cities. For example, in Beijing, traffic congestion has caused an economic loss of 105.6 billion Yuan (where environmental pollution loss of 4.52 billion Yuan), which equivalent to 7.5% of total GDP of Beijing (Beijing Traffic Development Annual Report, 2011). The

costs will be worse if the costs related to the heat island effects, waste pollution and environment deterioration are taken into account.

Therefore, to control traffic congestion has become one of the most important missions of the governments. There are many reasons for traffic congestion in Beijing, such as traffic demand along with socio-economic development, people's living standard improvement and tourist destination effect generated by the growth in the world and urban construction in Beijing. However, the most important reason is the unreasonable transport facilities structure and morphology in Beijing. Our study investigates the influence of the existing Railway Station distribution pattern in Beijing on urban traffic, and proposes a new model to build station distribution system based on high-speed railway station in Beijing in order to ease urban external transport function, relieve urban traffic congestion and guides urban spatial development. At last, we propose a general model of metropolitan station distribution.

2. THE STATUS AND PROBLEM OF BEIJING RAILWAY STATION DISTRIBUTION

Currently, the four largest railway stations in Beijing (Beijing Railway Station, Beijing North Railway Station, Beijing West Railway Station and Beijing South Railway Station) are all located nearby the Second Ring (see Figure 1). It seems to be scientific in early stage of development, because the Second Ring was the municipal boundary of ancient Beijing, so this railway station layout could meet the demand of urban rail transport under the premise of not affecting the inner city development pattern. However, with the rapid development of city, these stations are already in the central area of Beijing now, so the layout of the railway stations has brought many negative effects for the development of Beijing.

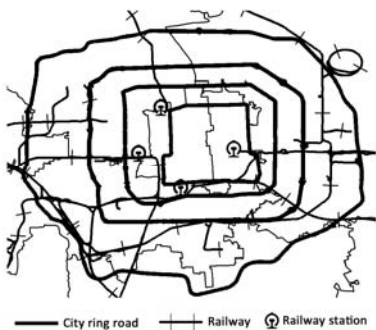


Figure 1. Position of Railway Station and City Ring Roads in Beijing.

2.1. Traffic pressure

Such a layout of railway stations will create enormous pressure for city center traffic. According to statistics, the total passenger volume of these four railway stations is about 50 million passengers per day in non-peak periods; while in the peak period, the volume will increase 1-fold, reaching approximately 100 million passengers per day. This huge external traffic flow becomes an important source of traffic congestion in city center.

Moreover, part of these traffic flows are unnecessary, some of the passengers who just need to reach the city's periphery (e.g. the Sixth Ring district) from outside Beijing, must be docked into the central area and then use local transportation to reach the city periphery.

2.2. Impact of railway facility

The major impact of railway facility on the city comes from railway stations and railway lines. Railway stations cover big area of city land, so cause interference to regional transportation organization. While, the impact of the urban railway lines is much greater than the railway station, it splits urban areas, occupies urban land resource and interferes the urban function of areas nearby.

2.3. High opportunity cost of land resources

Railway stations occupy a lot of valuable land in city center. According to statistics, Beijing four major railway stations occupy approximately 1,380,000 square meters (Beijing Railway Station 250,000 square meters, Beijing West Railway Station 510,000 square meters, Beijing South Railway Station 500,000 square meters, Beijing North Railway Station 120,000 square meters). Assuming converted into a commercial real estate with a plot ratio of 1.5, it will bring 7.452 billion Yuan (12.42 billion dollars) rental income every year (estimate by the average rent for commercial real estate inside the Second Ring -- per square meter per day 10 Yuan). So it's a big loss to occupy the higher differential rent urban centers areas as a transportation hub use.

3. RELEVANT RESEARCH REVIEW IN STATION DISTRIBUTION PLANNING

At the station distribution field, research mostly concentrated on small-scale transit line station spacing, and large-scale regional distribution of stations. While mesoscale research of station distribution in big cities is much less, the only study just explores the ideal model, focus on high-speed rail station distribution and

impacts of railway stations and railway lines on urban land use and spatial development.

Relationship between railway and city is complex, on the one hand railway can improve transportation accessibility, but on the other hand, railway also brings serious interference to the city, which is not primarily produced by the terminals themselves but the lines connected them. Therefore, it's an important issue to arrange the railway lines in the process of designing railway transport hub network system (Wang, 2008). Wang analyzes typical urban railway hub layout home and abroad, explores the coordination of railway layout and urban planning (Wang, 2007). With the rapid development of high-speed rail, the relationship between railway station siting and urban form and urban spatial structure has attracted more attention (Xu, 2009). Tian et al. (2011) conducts an in-depth research on the relationship between the location of high-speed rail station and urban spatial structure, analyzes the pros and cons of choosing the old city center, the airport or the new district as the site for high-speed rail station and then proposes that the location of high-speed rail station should be based on the city's long-term strategy, development capability and competitive advantage of the city and urban spatial structure.

4. NEW MODE OF STATION DISTRIBUTION IN BEIJING

4.1. Planning idea

It plans to establish a "C" type rail ring along the Six-Ring of Beijing, and lay out three integrated transport hubs in the south, east and north in Beijing, each hub consists of railway stations, long-distance bus terminals and bus stations (see Figure 2).

4.2. Planning content

4.2.1. Railway station

(1) Distribution

The landscape in the northwest Beijing is a low hill, east side is plain, considering the feasibility of the project, our study plans to built three high-speed railway stations in the south, east, north of Sixth Ring, forming a "C" type integrated railway hub around the city. Each station has the function of the current four railway stations. These three stations located evenly along the Sixth Ring.

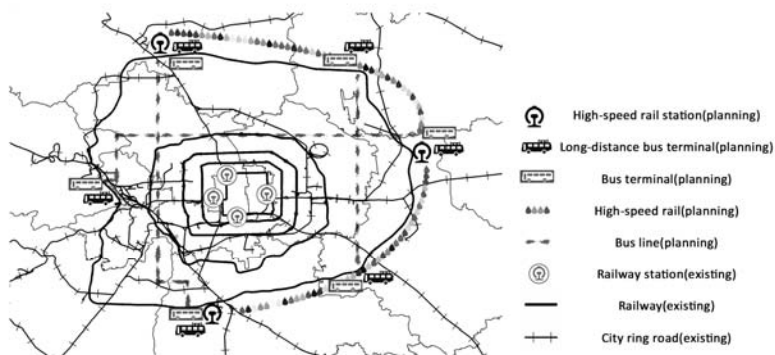


Figure 2. A New Pattern of Railway Station Distribution in Beijing

(2) Function

Three stations are all integrated stations, the service radius of them is about 30 km, and service area is more than 2000 square kilometers. Among them, the south station has the biggest size and works as a core portfolio hub, this is because Beijing Railway Station and Beijing West Railway Station are located in the south of Beijing, so it has a better basis of railway facilities, on the other hand, there are more population to the south of Beijing, so it will produce larger traffic. The other two stations are extension of the south station; they will serve the city passenger from east and north effectively, and avoid a lot of traffic through the inner city. Such distribution pattern of railway stations can progressively remove the railway lines inside Sixth Ring, thus achieving the goal of urban land resource optimization.

In addition, residents close to high-speed rail hubs can choose to take the ring railway to reach the other two high-speed railway stations, thereby avoiding the traffic flow through the city center and reducing the traffic pressure downtown.

4.2.2. Long-distance bus terminal

(1) Distribution

Long-distance bus terminals will be distributed evenly along the Sixth Ring. The distribution density will be denser than that of railway stations, but the three long-distance bus passenger transportation centers will still be located near to the three major railway hub stations, the other smaller terminals are evenly distributed along the lines connecting two railway stations (see Figure 2).

(2) *Function*

Each integrated transport hub contains long-distance bus terminal function, responsible for external long-distance bus transportation, especially for the passengers near to the railway station, it will offer them more choices among different types of vehicle.

4.2.3. *Bus terminu*

(1) *Distribution*

Bus terminals should be linked up with the high-speed railway transport hubs and long-distance bus hubs, also be distributed evenly along the Sixth Ring. Bus lines extend straightly into the city center, and then get to different places on edge of the city by other bus lines. Buses will not stay in city center half-way, and the parking management and maintenance of vehicles all happen around the Sixth Ring, all the bus lines eventually formed party mesh-state.

(2) *Function*

It will complete the connection of outside and inside transport, and form an inner-city transport network.

4.3. *Superiority of the new planning model*

The new station distribution planning model optimizes the overall urban rail transit system of Beijing, compared with the existing station distribution pattern, it has the following advantages (see Table 1).

Table 1. Compare between Existing and New Station Distribution

Items	Existing distribution patterns	New station distribution planning
Spatial	Near Second Ring	Near Sixth Ring
Pattern	Radial	Half-loop
Radiation range	Contains only one-way train from Beijing	Each direction into and out of Beijing
Functional Design	Just outside city transportation services	Serve outside city traffic mainly, also inner city traffic
Train Level	Low, medium, high-speed trains	High-speed trains
Traffic load	Great downtown traffic load	Traffic load distributed to the urban fringe
Opportunity cost	Take a lot of valuable land	Make use of cheap land in edge of city

5. AN IDEAL MODEL OF DISTRIBUTION OF STATIONS

5.1. Ideal model

5.1.1. Model hypothesis

The city is located in a homogeneous plain and has a large population and land size. Cities have Ring - Radiation road network structure, with the outermost ring as urban growth boundary. The transit system inside city is consist of rapid buses and has a network-like layout. Urban external rail traffic is all high-speed railway. In addition, the shape of city boundary is regulatory (cycle, oval, square, etc.).

5.1.2. Model Content

Metropolitan transportation hub is a composite style, making it possible to change between external and internal city transport. Transportation hubs are all located on urban growth boundaries, which can avoid passing-away traffic goes into the city center (see Figure 3).

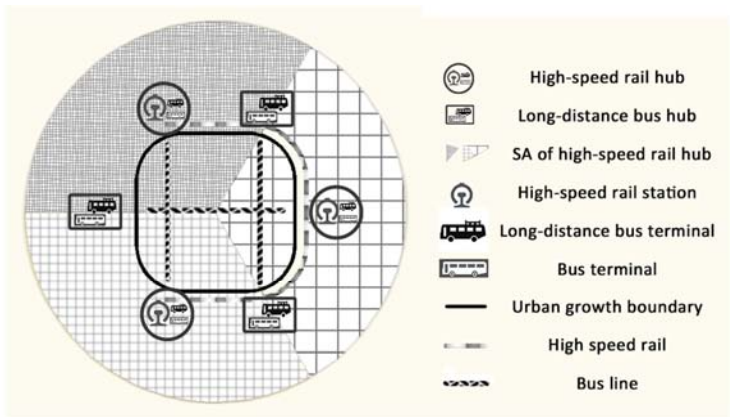


Figure 3. Ideal Model of Metropolitan Station Distribution

- ① Three high-speed railway hubs locate on the cycle evenly with the city center as the cycle core, each two high-speed railway hubs connect to each by the railway lines between them.
- ② Three long-distance bus hubs locate evenly between each two railway hubs.
- ③ Every high-speed railway hub can serve a fan-shaped service area (SA of high-speed rail hub) of the population, and the boundary of this area can extend to the periphery of the county and new town.
- ④ According to the actual size of the city and length of high-speed railway lines, long-distance bus hubs can be laid out more than one flexibly, but be careful to meet the requirements of uniform distribution.

⑤ Use BRT to accomplish the transit between external and internal city traffic, all the BRT lines totally form a grid look city public transportation system.

⑥ High-speed railway can also be used by the inner residents as a rounding railway. In order to ensure the efficient use of high-speed railway, only set up stations in three high-speed railway integrated hubs.

5.2. Model mechanism

Travelers are rational when make traffic use choices, they will be affected by distance and time and other factors.

5.2.1. Traffic flows: Outer city –City boundary

Traffic flow from outer city by means of high-speed rail or long-distance bus will make different choice according to the different destinations (see Figure 4).

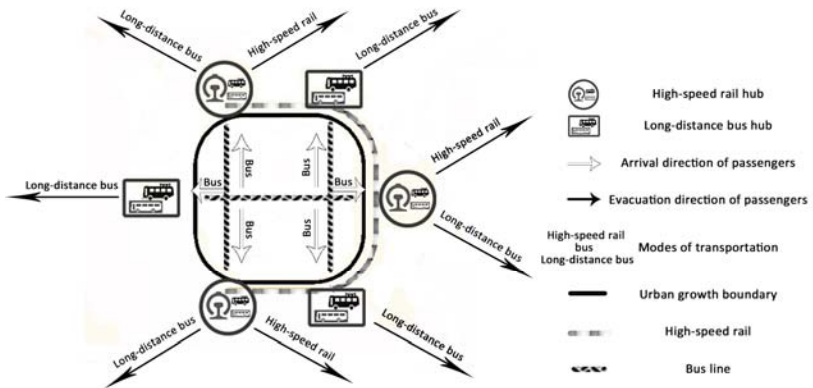


Figure 4. Passenger Flow and Transfer Mode Selection (From Outer City)

(1) Into city area

Situation 1: Enter the city directly by public transport;

Situation 2: Passengers whose trip destinations are at the other end of the city can choose to use the railway in the high-speed railway hub to get to another railway station, and then transfer to public buses into the city.

(2) Not into city area

Situation 1: Transfer to other high-speed rail lines or long-distance buses to the destination;

Situation 2: Passengers whose trip destinations are at the other end of the city can choose to use the railway in the high-speed railway hub to get to another railway station

5.2.2 Traffic flows: Inner city - City boundary

It's relatively simple of this situation, travelers arrive in the high-speed of long-distance transport hubs by public transportation, and then transfer to high-speed railways or long-distance buses to get to the outer-city destinations (see Figure 5).

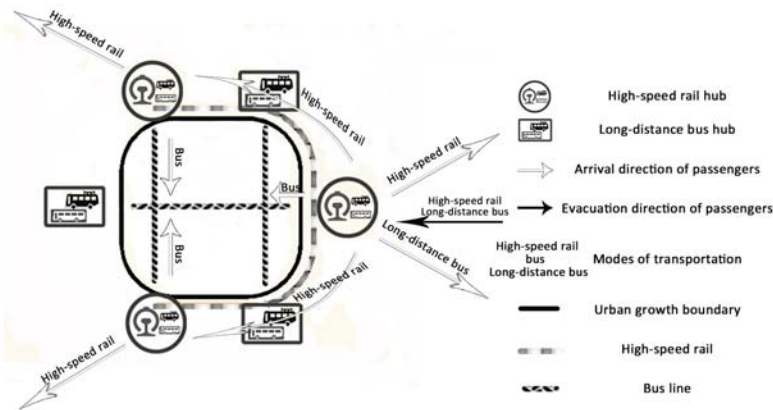


Figure 5.Passengers Flow and Transfer Mode Selection (From Inner City)

5.3 Model extension

Ideal model has strong applicability, for the cities which have plain landscape or those have not the suitable lands for large-scale railway construction in certain directions because of mountain terrain, water and other natural factors, this half-ring pattern of city combined rail hub mode will be worthy to be learnt from.

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An innovation in Green Transport for High-density Cities: the Rail transit-road System

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ABSTRACT

Traffic congestion relief has become a key issue in relation to building green transport in large cities in developing countries. In particular, in high-dense cities in the East Asia, traffic congestion is still getting worse even though huge investments have been made in transit and roads. One of important reasons for this is the disintegration of planning between road, transit and pedestrian systems. This paper introduces an innovation in green transport for high-density cities. It is named as rail transit-road system (RTRS). It bases on standard urban road system with introducing rail transit system on the roads. It gives the priority to green transport modes, such as pedestrians, cycling and public transport, at the same time taking cars and other vehicles into account. That makes it possible for cars, pedestrians, cycling and public transport to run efficiently without disturbing each other. Compared with existing roads, the RTRS system would enhance the efficiency of use of road spaces, save the investments, reduce traffic congestion on roads and improve the walking and bicycling environment. It would be an innovative way of achieving green transport in high-dense cities in the East Asia, for example, in China.

Keywords: Rail Transit, Road System, Design, Green Transport, Road Bridge, Intersection Bridge

1. INTRODUCTION

Traffic congestion relief has become a key issue in relation to building green transport in large cities in developing countries. Traffic congestion is caused by

the conflict over insufficient traffic infrastructure supply and rapid growth of traffic demand. Accordingly, congestion relief policies have been focused on the improvement of transport infrastructures on the one hand, and the control of travel demand on the other hand. In large cities in developing countries in Asia, huge roads and highway projects have been applied, for example, Beijing, Shanghai, Bangkok, Manila and Djakarta. However, in these cities, traffic congestion is still getting worse. One of major reasons for this is the disintegration of planning between road, transit and pedestrian systems. Another reason is that the investments in transit system largely lagged behind the investments in road systems.

Travel demand management is another important policy designed to relief traffic congestion. It aims to reduce the traffic demand by many tools, such as land use planning tools and VKT (vehicle kilometer travelled) control tools. VKT control tools usually include liming car license, raising parking fees, setting traffic congestion fees and so on. Travel demand management is often criticized as causing negative effects on people's basic need of mobility. In particular, in rapid growing cities in developing countries of Asia, strict travel demand control would hinder the economic development in the context of rapid urbanisation. It is also blamed for restraining the improvement of quality of life and created problems with social inequalities.

Therefore, the green transport strategy has been taken in many countries as a response to the disadvantages of previous policies. The strategy has been widely recognised over the past 20 years. It is focused on the enhancement of accessibility instead of the improvement of mobility (Lu, 2009; Pan, 2008). In the green transport system, car users, transit users, bicyclist and pedestrians can share transport spaces in a harmony way. There is a high integration between transit lines, road network and pedestrian and bicycling lanes.

Based on the principles of green transport strategy, the paper proposes an innovation in green transport for high-density cities. It is named as rail transit-road system (RTRS). It bases on standard urban road system with introducing rail transit system on the roads. It gives the priority to green transport modes, such as pedestrians, cycling and public transport, at the same time taking cars and other vehicles into account. In the RTRS system, a variety of transport modes including cars, pedestrians, cycling and public transport runs efficiently without disturbing each other. Compared with existing roads, the RTRS system would enhance the efficiency of use of road spaces, save the investments, reduce traffic congestion on roads and improve the walking and bicycling environment. It would be an innovative way of achieving green transport in high-dense cities in the East Asia, for example, in China. The following sections of the paper will introduce the RTRS in detail.

2. GREEN TRAFFIC THEORY

In order to control the negative effects of traffic congestion and traffic pollution, the scholars propose the concept of green transport. Along with a series of theories based on the concept of green transport, it gradually forms a relatively complete green transport theory system.

The concept of green transport system was first proposed on the Ottawa-Carleton Round-table Conference on the Environment by “Ottawalk” and Transportation Working Committee in 1992. Bradshaw (Bradshaw, 1994; TA, 2001) developed this concept, and think that a complete green transport system should be based on the principle of environment protection. The concept of “healthy city”, “humanized city” and “sustainable city” form the basis of the concept of green transport. “Healthy city” (WHO, 1994; Tsouros, 1995; Khosh-Chashm, 1995; Boonekamp et al., 1999) treats the human health as the core thing of urban planning and management. It requires the city to work in a good environment, and try to avoid demographic, social and economic negative externalities. In terms of the transportation field, negative externalities refer to traffic congestion, traffic accidents and traffic pollution. “Humanized city” is a people-oriented city which is ideal for walking and bicycle travel, the pleasant scale and dynamic environment will promote people's interaction (Gehl, 2010; Li, 2007). “Sustainable city” (Owens, 1996) is more concerned about the future of the city and harmonious development of society, economy and environment. In the transport field, the development and utilization of urban roads and energy resources all affect the ability of the sustainable development of city.

In the green transport strategy, different transport modes are given different levels of priority in terms of the recommend degree (support, encouragement, not encouragement, and limit). They ranked the priority as the follows order: walking, bicycle, public transport, service and freight vehicle, taxi, multi-passenger car and single-passenger car. In fact, different travel modes cannot be separated from each other. They are interacted and correlated. In practice, it is infeasible to manage different modes separately. The integration between different modes is a key issue of the implementation of green transport strategy.

3. DESIGN IDEAS FOR THE RTRS SYSTEM

The rail transit-road system is based on three key principles of green transport. They are health, humanization and sustainability. The RTRS system is targeted to solve three core problems of urban transportation: the traffic safety of travelers; the priority for public transportation; the efficient use of road network capacity. These three accomplishments provide a strong support for urban green transport development.

Rail transit-road system uses road bridges, intersection bridges and other three-dimensional interchanges means to achieve full closure of the road traffic BRT systems. In this system, the bus will follow a certain trajectory so it can be regarded as a quasi-rail traffic system. In this system, several central island style BRT stations or roadside style BRT stations will be set up on the road bridges. But only the central island style BRT stations will be used on the intersection bridges, with spiral staircase or vertical lift working as the connection of the upper and lower platforms, it forms a three-layer station structure of the basement, the ground layer and the upper layer. Therefore that makes it accomplishes zero transfer for passengers. The road system fully realizes the separation of pedestrians and vehicles, because pedestrians and vehicles are at different horizontal spaces, and it ensures the absolute safety of pedestrians. Also it uses the design of remote turning to accomplish the separation between left-turn vehicles and those from other directions.

4. DESIGN CONTENT FOR RAIL TRANSIT-ROAD SYSTEM

Rail transit-road system design includes many aspects, the most important ones are quasi railroad path and site design, and the most sophisticated are the road bridge design and the intersection design.

4.1 Road bridge

There is a road bridge every 200 meters in the Rail transit-road system considering the urban road landscape , the convenience and safety of pedestrians and road design standard, the total length of road bridge does not exceed 130 meters, this is there are not more than 3 road bridges per kilometer. This road system for Quasi railroad are consist of closed fast roads, and it's possible for small cars to accomplish remote turning. When talking about the width of the road, small cars turning radius is 6 meters, closed fast motor vehicle road is at least 16 meters wide(two-way four lanes),so the total width of the road is 24 meters.

(1) Design structure

The plan is going to set road bridges in non-intersections, each bridge has two layers (see Figure1), the ground layer is crosswalk (21), ground vehicle lane (2) and remote turning lane (22). The upper layer is two-way elevated lanes (3), set bus rapid transit station (34) at the middle of elevated platform, near to the buffer zone (32) or near the street, and get connected to the crosswalk through foot ladder side or lift (33).

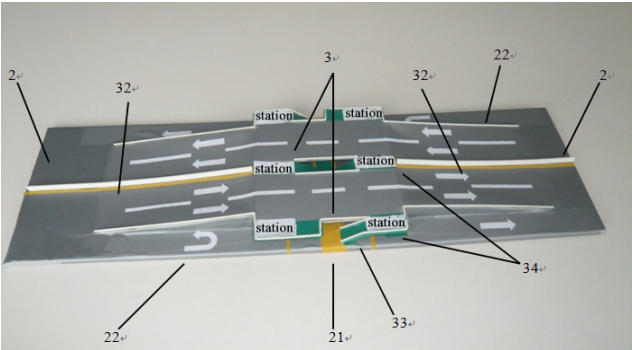


Figure 1. Schematic Diagram of the Road Bridge

(2) Design requirements

Escalator or stairs are needed for the convenience of passengers up and down. In order to prevent rain and snow, all stations are equipped with water-resistant devices.

(3) Design function

- ① It can achieve the separation of pedestrians and vehicular traffic. Pedestrians can go cross the streets without going through the pedestrian bridge, creating convenient walking environment.
- ② Using the potential and kinetic energy transformation mechanism, Bus Rapid Transit Station design can save energy.
- ③ It sets the parking space, convenient for cyclist to transfer. It sets parking spaces on both sides of the crosswalk, so people can transfer conveniently between bicycles and rapid bus, each parking space under the bridge will accommodate 50-100 bicycles.

4.2 Intersection bridge

(1) Design structure

The design at the crossroads includes three layers, (see Figure 2 and Figure 3): It's dug two-way trough type lanes (1), converged to the ground lane (2) or ground straight lane (23) via the ramp (11), in the middle with isolation zone (12). It's ground lane (2) in the Ground layer, and the ground straight lane (23), crosswalk bridge (21) and vehicle remote turning bridge (22). The second up-ground layer is overhead lane (3), it connected to the ground lane through 16 symmetrically distributed spans (31), and there are four in each respectively for upstream and downstream. It also set a rapid bus station (4) on the ground layer, and evenly places a station (34) in four directions, connects to the crosswalk bridge (21) through stairs (33). The station (34) via is located in the

middle of the road behind the stop line at the traffic lights, and there are also daylight openings (35) in the center of the crossroads.

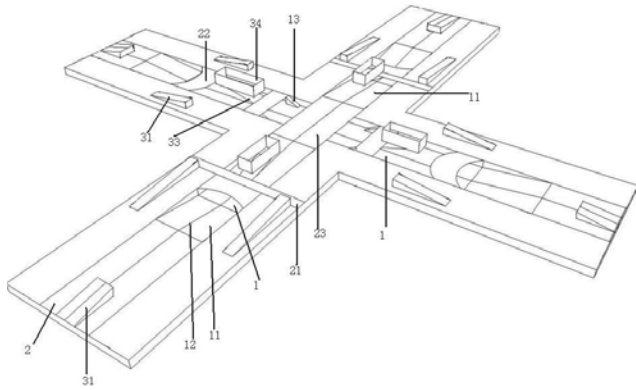


Figure 2. Schematic of Intersection Bridge Design (Ground Layer and Basement)

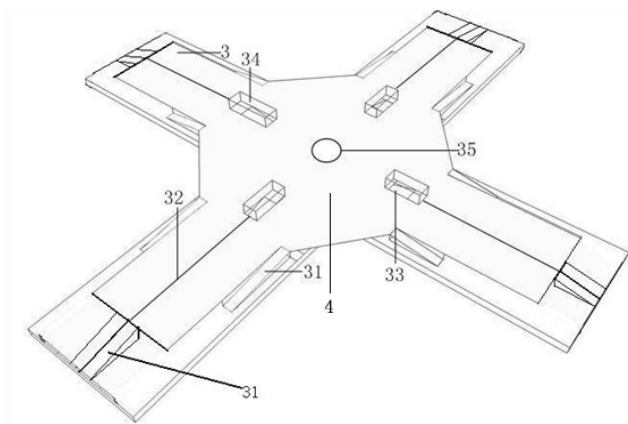


Figure 3. Schematic of Intersection Bridge Design (Second Up-Ground Layer)

(2) Design simplification

Intersection bridges can be simplified in practical application that is to discard the basement, and to retain the two-layer structure. Only pedestrians and non-motor vehicles can go through under the bridge, and then the bridge net height is 2.5 meters. The traffic lights in the 6crossroads Bridge Intersection need to be reequipped to separate lights for big cars and small cars: it means to maintain the big-size cars' traffic lights; Cancel the red lights and change the green lights to go straight or turn right indicators for small-size cars. The small-size cars can only go straight or turn right, and replace left turning by remote turning, so all small cars don't need to park and wait at the intersection, which can greatly improves the traffic efficiency at the intersections.

(3) Design requirement

According to the requirements of *urban road engineering design specification (CJJ37-2012)*, the net height of motor vehicles beneath the bridges at the intersections is 3.5 meters, the safe distance is 0.5 meters, the 3-meter height buses can pass under the bridge. Ground through lanes, crosswalks and remote turning roads are all at the same level. On the up-ground layer, bus stations are located evenly in the four directions. In order to prevent rain and snow, all stations are equipped with water-resistant devices. On the site, there will be the service of same-level embarkation, out-of-station car tickets selling and checking, real-time information monitoring system and station landscape architectural features to provide passengers with a safe and comfortable waiting environment with fast and convenient service on and off. There are buffer zones in the middle of the up-ground layers, daylight openings will be set at some distance to ensure enough light for dug two-way trough type lanes and ground lanes.

(4) Design achievement

① Public transport vehicles and large and small size vehicles can go up or down by the bridge approach set beside the up-ground lanes and turn right directly, without waiting for the traffic lights. This also avoids the conflict with pedestrians or other vehicles.

② There are pedestrian walking roads on the outer sides dug two-way trough type lanes and connected to the crosswalk bridge by stairs or lopes, used by pedestrians and non-motor vehicles to go across ground roads and also for the drivers on the dug two-way trough type lanes to escape from emergency.

③ This design ensures rapid buses only retain in the intersection of the up-ground layer, hypothetically speaking, it could accomplish no waiting in the intersection scientific and rational traffic lights design.

④ This Rail transit-road system solves the problem of crossing the roads when passengers want to go inside and out of the stations. It reserves the innermost lane for public bus use only in order to accomplish bus priority. At the same time, it achieves two-way remote turning for small cars, so solves the left turning problems in the intersection for small cars and reduce the traffic pressure there.

5. APPLICATION EFFECT OF RAIL TRANSIT-ROAD SYSTEM

5.1 Advantage

(1) Walking-friendly, travel means structure optimization

The design aimed for pedestrians and non-motor design is a major bright spot. In this design, both pedestrians and non-motor vehicles who want to go cross the street or through the intersection always walk on the ground, while the motor vehicle on the bridge. It achieves the pedestrian-friendly philosophy. All road intersection designs also give the ground path back to the pedestrian and

non-motor traffic; it will attract more travelers to choose walking and cycling, and improve the travel structure.

(2) Improvement of transport efficiency

The design of rail transit-road system gives the priority to the walking, cycling and public transport, taking the passage of other vehicles into account. It achieves the goal that pedestrians, bicycle traffic, public transport and small car traffic go smoothly, without disturbing each other or occupying each other's space. Although there are traffic lights at the intersection, but without the interference from pedestrians and other non-motor vehicles, through the computer optimization of the traffic lights. Vehicle running speed will not be affected by traffic lights in theory. Compared with the existing road system (see Table 1), road capacity and utilization has reached maximum value.

Table 1. Compare between Railroad System, the Existing BRT System and Rail Transit-Road System

	Railroad system	Existing BRT system	Rail transit-road system
Cost and construction period	High cost and long construction period	Moderate cost and long planning horizon	Moderate cost and no pre-planning period
Passenger capacity and travel speed	Largest transport capacity fastest	Medium faster transport capacity, fast	Large transport capacity fastest
Pedestrian crossing and security	Unrelated	Unrelated	pedestrians and non-motor vehicles' safety and convenience
Passenger convenience	General	General	Very convenient
Impact on traffic congestion	a certain role in mitigation	a certain role in mitigation	Effectively alleviate traffic congestion
Whether universal	No	No	yes
whether replace ordinary bus	No	No	yes
Impact on the urban landscape	Metro has no effect, elevated rail transportation has effect	Elevated BRT influential	No effect
Impact on other vehicles	Unrelated	Greater impact on other vehicles	Cooperate with other vehicles

(3) Conservation of land resources

The bridges in this system are all small and medium sizes, and are all lower than 4.5 meters, so they will not affect urban landscape. If remake the Siyuan Bridge in Beijing with the bridges in this system, the area for bridges will reduce to 20 million square meters to the current 50 million square meters, which will save a lot of urban land resources. This design is also applicable to other cities' large bridge group transformation.

(4) Good applicability

This design will reduce the limitation of the road width for rapid buses, thus saving land resources occupied by province-level roads.

(5) Landscape and ecology optimization

The bridge height in this system is lower than the existing one, which makes urban landscape more harmonious. It meets the requirements of green transport and lay a foundation for the Ecological city construction.

(6) Cost saving

Rail transit-road system can replace the subway and rail transportation, it also occupy an absolute advantage in terms of the cost of construction.

(7) Urban transport guide urban development

To achieve the full cover of rail transit-road system and stick to walking, bicycle and public transportation oriented development model. It will make the city development more balanced and the public service facilities more equal.

5.2 Rail transit-road system's application in Beijing

It's about 60 km from north to south and from east to west inside the Sixth Ring in Beijing, if equated one railroad system with one kilometer as interval, we need to transform about 7,000 km of roads. Each number of buses has its own road line, and the total number of lines will reduce to 120 or so from the current more than 900, and to achieve full coverage of BRT. We can transform the city ring roads with ground bridges at first, so try to achieve the full cover of rail transit-road system in Beijing.

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Street Names and the History of Hong Kong Island

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ABSTRACT

The German philosopher Emmanuel Kant once said that there were only two bodies of knowledge in the world, viz. history and geography. Both studies reflect on time and space, and one cannot be separated from another, history has to be studied in the context of geography. The purpose of this paper is to use the names of streets on Hong Kong Island to provide a glimpse of the history of Hong Kong. Hong Kong represented the first time in modern history where a western power has officially ruled over a predominantly Han Chinese population. Prior to the colonization of Hong Kong, the Dutch have occupied Taiwan (but only for 38 years and the rule was over the local aborigines). While Macao was occupied by the Portuguese much earlier, the occupation was not official, and Macao was not declared a colony until after the first opium war, when the Portuguese took advantage of the weak Qing government to press their demands.

INTRODUCTION

The German philosopher Emmanuel Kant once said that there were only two bodies of knowledge in the world, viz. history and geography. Both studies reflect on time and space, and one cannot be separated from another, history has to be studied in the context of geography.

The traditional Chinese belief of human activities focuses on four aspects – clothing, food, lodging and transportation. Nothing reflects transportation better than roads and streets, and names of places including roads and streets reflect how people actually lived. In other words, names of places are fossils that record human activity in a geographical location. Examples can be found across the world. The Philippines, for instance, was named after Philip II of Spain. The State of Louisiana was named after Louis XVI of France. In many cities, street names often depict activities that once thrived. In Liverpool, the main railway station is located at Lime Street, a place where lime kilns once lined the street. In Beijing, many places were named after landmarks. Just to mention a few examples: Vegetable Market Junction (菜市口), Lantern Market Junction (灯市口), etc. It is therefore not surprising that on October 17, 2003, geographical names have been designated by the United Nations Education, Scientific and Cultural Organization as within the meaning of the Convention for the Safeguarding of the “intangible cultural heritage”. Resolution VIII/9 recognized the importance of geographical names as part of a nation’s

historical and cultural heritage and Resolution IX/4 recognized that geographical names are part of the intangible cultural heritage.

The purpose of this paper is to use the names of streets on Hong Kong Island to provide a glimpse of the history of Hong Kong. Hong Kong represented the first time in modern history where a western power has officially ruled over a predominantly Han Chinese population. Prior to the colonization of Hong Kong, the Dutch have occupied Taiwan (but only for 38 years and the rule was over the local aborigines). While Macao was occupied by the Portuguese much earlier, the occupation was not official, and Macao was not declared a colony until after the first opium war, when the Portuguese took advantage of the weak Qing government to press their demands.

HISTORY OF HONG KONG

The territory of Hong Kong consists of 3 major areas – Hong Kong Island, Kowloon Peninsula and New Territories. The Island of Hong Kong was officially ceded to Britain in 1842 upon signing of the Treaty of Nanjing (although the British has occupied the Island as early as 1841) as a result of the infamous first opium war. The Kowloon Peninsula was ceded to Britain in 1860 following the Treaty of Beijing as a result of the second opium war. The New Territories (consisting of the land adjoining to Mainland China and the surrounding islands) was leased to Britain in 1898 for a term of 99 years.

In terms of land mass, the entire territory of Hong Kong is only 1,100 square kilometers. The Island of Hong Kong, at 80 square kilometres, stands for only 7.3% of the land mass (while the Kowloon peninsula is 47 square kilometres or 4.2% and the New Territories plus the remote islands is 969 square kilometres or 88.5%). However, the island of Hong Kong is more important because it was the first part of the colony to be occupied and hence has a longer history. It is also the seat of the government and the major trading houses. The street names in Hong Kong represent a good reflection of the interaction between the eastern and western cultures.

SIGNIFICANCE OF MILITARY AND MILITARY FIGURES

The significance in the military influence is reflected in the fact that the Commander of the British Forces got his official residence built nine years before that of the Governor of Hong Kong. The military (consisting of the navy and the army) was considered very important for the survival of the colony because Hong Kong was won by military might, and hence there was always a threat for the Qing government to reclaim the territory by force. There were also hostile western powers such as the Dutch and the Portuguese who threatened to take over the territory. The place was also infested by pirates (and stories of these pirates still form part of the local folklore). The military held significant portions of land in Hong Kong because the boundary was changed every 30 years (1842/1860/1898), and the land facing the Qing border had to be guarded. Once land is occupied by the military, it was difficult for the Colonial Ministry to get it back.

The first occupation of Hong Kong took place in 1841, when Captain Belcher, the commander of the warship HMS Sulphur, first landed in Hong Kong. The troops landed in Sheung Wan where the British forces staged a parade. The names within that area still bear their footprints. The channel between Hong Kong Island and the Green Island where the HMS Sulphur first sailed through was named the Sulphur Channel (硫磺海峡). Belcher Street (卑路乍街) was named after Captain Belcher at a location where he built the first battery to guard the harbor – Belcher Battery. His landing point also sheds some interesting clues as to how the Chinese and the English names are interpreted, even to this date. He landed at a place called Bonham Strand but the Chinese names (文咸東街 and 文咸西街) made no reference to the beach, which was later reclaimed in 1851 for buildings to accommodate Chinese trading houses. The parade ground is still called Possession Point in English, but the Chinese name is 水坑口, which translates into creek mouth. It can be imagined that it must have been the sandy shoreline and the fresh water supply that lured Captain Belcher to choose that landing spot.

There are other places in Hong Kong that still bear the names of the warships that used to be stationed in Hong Kong. Some famous ones include the HMS Repulse (now the tourist spot Repulse Bay) and the HMS Tamar (now the garrison headquarters of the Chinese armed forces in Hong Kong).

Other places also reflect its military past. Examples include Sai Ying Pun (西營盤), Arsenal Street (軍器廠街), Admiralty (金鐘), Battery Path (炮台里).

There were not many British subjects in Hong Kong. The civilians were busy with trading, the missionaries had their own goals, and there were only a few colonial staff. It was not surprising that many of the early government officials were drawn from the military. Many of the streets testified to these people. Some examples include:

- Caine Road in Mid-levels next to the prison – after Captain William Caine from the Army, the first Attorney-General and Police Chief
- Peddar Street in Central at the waterfront before reclamation – after Captain William Peddar from the Navy, the first Port Master
- Collinson Street in West Point and Cape Collinson in Chai Wan – after Captain Collinson, who as an army engineer, made the first detailed survey on the land on Hong Kong Island
- Elgin Street in Mid-levels – after Lord Elgin who as Commander of British-French Forces in the “Arrow War”, also represented Britain in the ceding of Kowloon on January 19, 1861
- Seymour Road in Mid-levels – after Admiral Seymour who served as Admiral for British/French Joint Forces
- Harcourt Road in north shore of Hong Kong Island next to the naval base – after Rear Admiral H.J. Harcourt who landed in HK to take over Hong Kong from the defeated Japanese on September 16, 1945. He formed the military government and ruled over Hong Kong until the handover to civilian government led by Governor Mark Young 9 months later.

THE ROYAL HOUSEHOLD

Britain was and still is a constitutional monarchy. Everything was done in the name of the emperor or the empress. The Hanover-Windsor House of royalty is famous for their longevity. Throughout the 150 years of colonial rule, there were only 6 rulers (including the short-lived rule of Edward VIII who abdicated to marry a divorced woman):

- Queen Victoria (1837 – 1901)
- King Edward VII (1901 - 1910)
- King George V (1910 – 1936)
- King Edward VIII (1936)
- King George VI (1936 – 1952)
- Queen Elizabeth II (1952 – present)

Queen Victoria ruled over the British Empire at her zenith, and she ruled for 64 years. Throughout the British Empire, places big and small were named after her – Victoria Falls, the state of Victoria, etc. Hong Kong first became a British colony during her reign and it was not surprising the find major streets named after her. Hence we have:

- Queen’s Road East, Central and West (皇后大道西, 皇后大道中, 皇后大道東)
- Queen’s Street (皇后街)
- Queensway (金鐘道)
- Victoria Road (域多利道)
- Upper and Lower Albert Road (上亞厘畢道, 下亞厘畢道) – after Prince Albert, the husband of Queen Victoria

When the roads were named, a corresponding Chinese name was also adopted for the Chinese population. However, when the translator undertook the task, he was unaware of the fact that the ruler of such a big empire was a woman, and he took it for granted that the “queen” for Victoria stands for the “consort queen”, and the Chinese term consort queen (皇后) was chosen, instead of the more correct term for queen empress (女皇). Therefore up to this date, the mistakes still stand and the Chinese names still infer the queen consort. Such mistakes were not uncommon in the early days when people did not have bilingual skills. An example is the name Alexander. As Chinese read from right to left instead of left to right for the British, the name Alexander was written as Rednaxela, which still stands today in the Mid-levels.

Of course, there are other roads dedicated to subsequent rules and royal family members, hence there are names such as King’s Road, Great George Street and Connaught Road.

Worth mentioning is the Statue Square in Central which again was wrongly translated as the Queen’s Square in Chinese (皇后像廣場). Called the Royal Square before the Second World War, it used to house Queen Victoria (in the centre) with her family members. During the war, the Japanese took all the statues back to Japan to be smelted. Only some have been recovered and the statue of Queen Victoria was re-instated in the Victoria Park. Hence the “Queen Statue Square” no longer has the statue of the Queen.

WHO WERE THE HONG KONG RULERS?

In his book *HK Borrowed Place and Borrowed Time* (1976), Richard Hughes said that the true rulers of Hong Kong were (in order of importance), the Jockey Club, Jardine, Hong Kong Bank and the Governor of Hong Kong.

The truth of his claim may be found in the membership of the Executive Council (equivalent to the Cabinet in western countries) in 1982. The Council was chaired by the Governor. Amongst the members were the Jardine taipan and the HK Bank taipan who traditionally were always allocated a seat in the Executive Council, the seat of power in Hong Kong. The Jockey Club enlists all the elite members in the society, and anyone of any influence in Hong Kong is a member of the Jockey Club. In this case, all members of the Executive Council were members of the Jockey Club.

GOVERNORS AND HIGH OFFICIALS

As the representative of the British Crown, the governors had enormous powers. It is not surprising that of the 28 governors, 19 of them have at least one road named after him. These include:

- Sir Henry Pottinger (砵甸乍爵士) 1843 – 1844
 - Pottinger Street
- Sir John Francis Davis (戴維斯爵士) 1844 – 1848
 - Davis Street, Mount Davis
- Sir Samuel George Bonham (文咸爵士) 1848 - 1854
 - Bonham Strand, Bonham Road
- Sir John Bowring (寶靈爵士) 1854 – 1859
 - Bowrington Road, Bowring Street
- Sir Hercules Robinson (夏喬士·羅便臣爵士) 1859 - 1965
 - Robinson Road
- Sir Richard Graves Macdonnell (麥當奴爵士) 1866 - 1872
 - Macdonnell Road
- Sir Arthur Edward Kennedy (堅尼地爵士) 1877 - 1883
 - Kennedy Road, Kennedy Town, Kennedy Street
- Sir John Pope Hennessy (軒尼詩爵士) 1877 - 1883
 - Hennessy Road
- Sir George Ferguson Bowen (寶雲爵士) 1883 - 1887
 - Bowen Road
- Sir George William Des Voeux (德輔爵士) 1887 - 1891
 - Des Voeux Road
- Sir William Robinson (威廉·羅便臣爵士) 1891-1898
 - No road (as there was already another Robinson)
- Sir Henry Arthur Blake (卜力爵士) 1898 - 1903
 - Blake's Pier, Blake's Garden
- Sir Mathew Nathan (彌敦爵士) 1904 - 1907
 - Nathan Road
- Sir Frederick Lugard (慮押爵士) 1907 - 1912

- Lugard Road
- Sir Francis Henry May(梅含理爵士) 1912 - 1919
 - May Road
- Sir Reginald Edward Stubbs(司徒拔爵士) 1919 – 1925
 - Stubbs Road
- Sir Cecil Clementi(金文泰爵士) 1925 – 1930
 - Sir Cecil's Ride, Lady Clementi's Ride
- Sir William Peel(貝璐爵士) 1930 – 1935
 - Peel Street
- Sir Andrew Caldecott (郝德傑爵士) 1935-1937
 - Caldecott Road in Kowloon
- Sir Geoffrey Alexander Stafford Northcote(羅富國爵士) 1937 -1941
 - No road

Except for Caldecott Road, all the other streets were on Hong Kong Island. No street name has been assigned to all the governors listed below after the War

- Sir Mark Aitchison Young (楊慕琦爵士) 1941 – 1947
- Sir Alexander William George Herder Grantham (葛量洪爵士) 1947 - 1958
- Sir Robert Brown Black (柏立基爵士) 1958-1964
- Sir David Clive Crosbie Trench(戴麟趾爵士) 1964-1971
- Sir Murray Maclehole (麥理浩爵士) 1971-1982
- Sir Edward Youde (尤德爵士) 1982-1986
- Sir David Wilson(衛奕信爵士) 1987-1992
- Rt. Hon. Christopher Francis Patten(彭定康爵士) 1992-1997

In the old days where there was no aviation, it took a long time for governors to come to work from Britain or other British territories. Many of the governors also fell sick due to the different weather in the colonies. Their absence from their position could be longer than a year, and hence there were also a number of acting governors chosen from the government officials (mainly the Colonial Secretary or the Commander of British Forces). Some examples include:

- Major-General William Jervois 1851
 - Jervois Street (乍畏街 or 蘇杭街) in Central
- William Mercer 1865-66
 - Mercer Street (孖沙街) in Central
- Major-General Henry W. Whitfield 1870
 - Whitfield Road (威非路道) in Causeway Bay
- William Marsh 1882-3, 1885-7
 - Marsh Road (馬師道) in Wanchai
- Major-General WG Cameron 1887
 - Mount Cameron (金馬麟山道) on Hong Kong Island, Cameron Road (金馬倫道) in Tsim Sha Tsui
- F Fleming 1890
 - Fleming Road (菲林明道) in Wanchai
- Major-General Digby Barker

- Barker Road (白加道) on the Peak
- Major-General William Black 1898
 - Black's Link (布力徑) on the Peak
- Claude Severn 1912, 1913, 1917, 1922
 - Severn Road (施勳道) on the Peak
- Rear Admiral Cecil Harcourt 1945-46
 - Harcourt Road (夏愨道) in Admiralty

There are also senior government officials such as Attorney-General, Treasurer, Director of Public Works, Chinese Secretary, etc. where street names were named after the incumbents.

JARDINE AND OTHER FOREIGN HONGS

It was often said that Jardine Matheson was the prime mover behind the opium war and the occupation of Hong Kong. Hence Jardine was one of the first to bid for land in Hong Kong for its business activities. The key turf was East Point (now Causeway Bay). A lot of streets were named after Jardine Matheson and the top management (the taipans). Examples include:

- Jardine's Bazaar (渣甸街)
- Jardine's Crescent (渣甸坊)
- Yee Wo Street (怡和街)
- Sugar Street (糖街)
- East Point Road (東角道)
- Paterson Street (百德新街)
- Cannon Street near where they fly the noonday gun (景隆街)
- Percival Street (波斯富街)
- Matheson Street (勿地臣街)
- Keswick Street (敬誠街)
- Irving Street (伊榮街)
- Jardine's Lookout (渣甸山)

Compared with Jardine, Hong Kong Bank was a latecomer but the influence must be not under-estimated as it also serves as the Central Bank in Hong Kong. The influence is reflected in the Statue Square, where the statue of Sir Thomas Jackson, the most successful taipan in the history of Hong Kong Bank, still stands. It is the only remaining statue in the square where it once rubs shoulders with the royal family. The road next to the square was called Jackson Road.

Many streets in Hong Kong were named after the other trading houses or the names of the owners. For example, there was Gilman Street in Central after Gilman and Co. There was also the cluster of warehouses in Wanchai named after their owners, and the streets were named after them – Heard Street, Malory Street, Burrows Street.

CHINESE STREETS

The population of Hong Kong grew as it became more and more turbulent in China, which explains the reason behind the cluster of streets in Central to name after auspicious wishes. These include:

- Wing Wo Street (永和街) – street of eternal harmony
- Wing Shing Street (永勝街) – street of ever victory
- Wing Lok Street (永樂街) – street of perpetual happiness
- Wing Kut Street (永吉街) – street of perpetual fortune
- Wing On Street (永安街) – street of eternal peace

Hong Kong started off as a fishing port and an entrepôt; both trades are closely related to the maritime industry – a risky business because of the low technology and the unpredictable natural elements. The traditional gods associated with maritime safety were Kuan Yin (觀音), Heavenly Goddess or Mazhu (天后, 媽祖), Hung Shing (洪聖) and Tam Kung (譚公). Many of the streets located on the shoreline or what used to be the shoreline were named after these gods. Examples include:

- Hung Shing - Hung Shing Temple, Tai Wong East Street, Tai Wong West Street (洪聖古廟, 大王東街, 大王西街)
- Tam Kung - Tam Kung Temple Road, Ah Kung Ngam (譚公廟道, 阿公岩)
- Kwun Yam - Kwan Yam Shan, Lin Fa Kung Street (觀音山, 蓮花宮街)
- Tin Hau - Tin Hau Temple Road (天后廟道)

As people settled down, Chinese businessmen also made their fortune and they eventually owned an entire street. Some of the prominent Chinese businessmen and their streets were:

- Li Sing Street – Li Sing (李陞街)
- Li Chit Street – Li Chit (李節街)
- Lee Garden Hill, Lee Garden Road – Hysan Lee (利園山道)
- Lee Po Lung Road / Lee Po Lung Path – Lee Po Lung (李寶龍路)
- Shouson Hill Road – Shouson Chow (壽山村道)
- Lee Yuen East and West Street - Kam Lee Yuen (利源東/西街)
- Chun Yeung Street - Kwok Chun Yeung (春秧街)
- Po Shan Road - Wai Po Shan (寶珊道)
- Lan Kwai Fong - Chan Kwai Hon (Alias Lan Heen) (蘭桂坊)

CONCLUSION

Compared with the scientific naming system in North America (Fifth Avenue, Sixth Avenue, etc), the street names associated with traditional towns and cities definitely offer more colour, and more information about the past. In the case of Hong Kong, it definitely offers a glimpse of the interaction between the Chinese culture and the western culture. From an historical point of view, it was appreciated that the government has shown its tolerant attitude that even after the handover of the sovereignty, there was no attempt to replace the colonial street names with more politically correct names. After all, the colonial past was also part of the history of Hong Kong.

Research on Installation Site of Loop Detectors for Link Travel Time Estimation

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ABSTRACT

Installation site of loop detectors directly influence the quantity and quality of information detected and processed. Searching appropriate installation site not only contribute to realize more accurate traffic estimation, but also is able to accomplish a better cost-effectiveness. In this paper, three positions were chose as the loop location alternative sites: site beside upstream intersection, site beside downstream intersection and sites in middle of link. By comparing their average relative errors (AREs) and relative AREs distribution of link travel time, a better loop location site could be determined on the basis of definition of link travel time in HCM 2000. Moreover, different volumes in road network were also considered to test the outcomes. VISSIM simulation was used to build the road network in order to detect all the traffic data. The purpose of this paper is to seek superior locations of loop detectors for dynamic link travel time estimation in urban road.

Keywords: Loop Detector, Installation Site, Link Travel Time

1. INTRODUCTION

Effective surveillance of traffic condition is the premise to realize dramatic traffic control and management. As one of the most widely deployed type of traffic sensors, loop detector plays a vital role to detect the basic traffic parameters of vehicles passing through, including traffic flow, instantaneous speed and occupancy rate, by installing a single induction loop per lane every 500m to 2km in freeways or around intersections in urban road network. Considering the superiority of loop detectors in veracity and universality, it still is the main trend in dynamic traffic information collection.

Loop detector is still one of the most feasible and cost-effective methods to estimate link travel time, an important index to quantifying traffic system performance, by using the information like traffic volume and occupancy (Yaojan *et al.*, 2012).

Many other statistical methods like Kalmanfilter, Markov chain Monte Carlo (Martinet *al.* 2004) were applied to improve the link travel time estimation.

However, fewer studies emphasize on the installation sites of fixed detectors. Although Adam *et al.* (2001) presented a mixed-integer linear program to optimize the locations of loop detectors in freeway, but the influence of installation sites in urban roads on link travel time was barely paid enough attentions. Moreover, a large amount of damage of loop detectors was caused due to frequent acceleration and deceleration of vehicles around intersections. Therefore, this paper focused on installation position of single-loop detectors, and tried to find an alternative site for loop detectors to avoid the dead-weight loss. Three sites were chose including beside upstream intersection, site beside downstream intersection and sites in middle of link as the study positions. The purpose of this paper is to seek the superior location for estimating link travel time in urban roads.

Four sections were organized in this paper. Based on the estimation of link travel time in the 2nd section, estimation results of different installation sites and relative analysis based on different volumes showed in section 3. The conclusion was depicted in section 4.

2. LINK TRAVEL TIME

Link travel time is one of the most significant parameters to measure the traffic conditions of urban road network. Hence, kinds of methods to estimate link travel time from single-loop outputs were proposed. Jin *et al.* (2010) proposed an estimation method of link travel speed based on Bayesian network method by analyzing the relationship between volume, occupancy and speed. The results calculated proved the efficiency of the method to reduce the mean absolute error of speed. Benjamin (2009) tried to refine the non-conventional techniques using moving median method, sequence method and distribution method for estimation speed at freeway from single-loop detectors in order to approach the accuracy of a dual-loop detector's measurement. Using dual-loop detector to estimating link travel speed/time was widely noted as well. A Bayesian analysis is performed after investigating statistical inference for vehicle speed from dual-loop detectors by Baibing (2010).

Considered that finding the superior location is the ultimate target, an original method based on the definition of link travel time was chose to estimate link travel time, which could avoid influences from the improvement methods themselves.

According to HCM 2000 (National Research Council), link travel time was defined as

$$T = T_R + T_S \quad (1)$$

Where,

T_R : running time of a link,

T_S : stopping time of the downstream intersection.

Both T_R and T_S could be calculated by the information detected by loop detectors. Assume that v , was the average running speed in the link

$$T_R = D / v \quad (2)$$

$$T_s = d_1 + d_2 \quad (3)$$

$$d_1 = \frac{c(1 - \frac{q}{c})^2}{2(1 - \min[1, x] \frac{q}{c})} \quad (4)$$

$$d_2 = 900T[(x-1) + \sqrt{(x-1)^2 + \frac{8ex}{CAP \cdot T}}] \quad (5)$$

Where

D : the length of the link,

d_1 : delay generated from uniform arrival of vehicles,

d_2 : random addition delay.

Since loop detectors could only collect vehicles' instantaneous speed, treating collected speed as the average running speed may cause extra errors. Hence, Bayesian Estimation was used to estimate the average running speed on the basis of original data samples.

The basic thought of Bayesian estimation is to estimate the value of parameter μ if some other additional information of this unknown parameter is provided. Estimated value could be obtained based on the following formula (Hua and Xiaokun, 2005).

$$p(\mu | x_1, x_2, \dots, x_t) = \frac{p(\mu; x_1, x_2, \dots, x_t)}{p(;; x_1, x_2, \dots, x_t)} \quad (6)$$

Where,

μ : estimated value, which is following $N(\mu_0, \sigma_0^2)$

X_k : detected value, which is $N(\mu, \sigma_k^2)$

Then, the estimated value $\hat{\mu}$ could be expressed as

$$\hat{\mu} = \left[\sum_{k=1}^t \frac{x_k}{\sigma_k^2} + \frac{\mu_0}{\sigma_0^2} \right] / \left[\sum_{k=1}^t \frac{1}{\sigma_k^2} + \frac{1}{\sigma_0^2} \right] \quad (7)$$

3. ESTIMATION RESULTS BASED ON DIFFERENT SITES

A simple road network was built by VISSIM. Considering the vulnerability of loop detectors in reality, long roads were designed in simulation with lengths from 600meters to 1000 meters, and 10 identified links and 3 installation sites a, b, c were chose as study objectives showed in figure 1. The traffic signals of all the intersections were assumed to have only 2 phases with the cycles of 120 seconds, and intersection 1 and intersection 6 are using signal timing 1, while others are using signal timing 2.

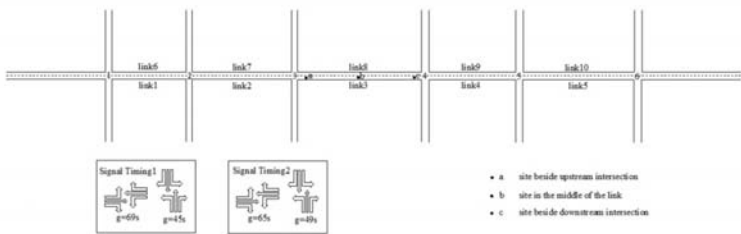


Figure 1. Simulation Road Network.

The average relative error (ARE) could directly reflect the accuracy of outcomes, which could expressed as

$$ARE = \frac{T_i - T_f}{T_f} \tag{8}$$

Where,

T_i : estimated results,

T_f : true value.

The original set of volume was 1200 pcu in arterial roads, and 600pcu in collector roads. After 20 times running ,the AREof link travel time estimated from different installation sites was showed in table 1.The columns belonged to collected instantaneous speed are the AREs calculated by substituting average running time with vehicles' instantaneous speed, while the columns belonged to Bayesian estimation speed stand for the corresponding results with the collected speed being processed by Bayesian estimation.

Table 1.The AREs from Different Installation Sites.

Direction	Link Number	Collected instantaneous speed			Bayesian estimation speed		
		Site a	Site b	Site c	Site a	Site b	Site c
From Left To Right	1	0.086	0.085	0.247	0.088	0.086	0.116
	2	0.055	0.050	0.373	0.056	0.051	0.120
	3	0.043	0.046	0.522	0.042	0.046	0.227
	4	0.064	0.060	0.416	0.065	0.061	0.113
	5	0.058	0.054	0.333	0.055	0.052	0.128
From Right To Left	6	0.090	0.087	0.475	0.091	0.088	0.078
	7	0.055	0.050	0.473	0.055	0.050	0.134
	8	0.044	0.045	0.418	0.045	0.046	0.201
	9	0.058	0.053	0.394	0.057	0.053	0.134
	10	0.061	0.056	0.400	0.063	0.057	0.107
Average ARE		0.061	0.058	0.405	0.062	0.059	0.136

It could be easily found the regularities of different installation sites and

speed patterns.

(1) Different installation sites: AREs estimated from site c are way more larger than that of the other two sites. Besides, calculation results of site a and site b have the same error level, and site b could be considered as a relative better installation site.

(2) Different speed patterns: Bayesian estimated speeds cannot influence the AREs too much at site a and site b, which reflects vehicles' instantaneous speed at site in the middle of a link and site beside upstream intersection is close to the average running speed. However, Bayesian estimation could obviously improve the accuracy of results when the installation sites are at site c. The ARE decline from 0.405 to 0.136.

The distribution of ARE in Figure 2 could also reflect the regularities relatively. Fit 1, fit 2 and fit 3 represent to distribution fitted by AREs estimated from site a, site c and site b separately based on Bayesian estimation speed, while fit 4, fit 5 and fit 6 stand for that based on collected instantaneous speed.

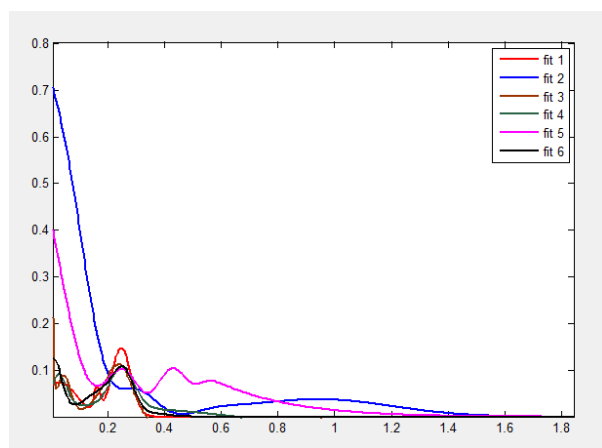


Figure 2. Distribution of ARE.

From figure 2, the distribution curves of fit 2(the blue curve) and fit 5(the pink curve) are distinguished which represent to the distributions at site c. Although the highest probabilities centralize in 0, the AREs which are larger than 0.8 have a relative higher probabilities. Since fit 2 tends to 0 more intensively than fit 5, the advantage of Bayesian estimation speed in fluctuation data samples could be demonstrated. Besides, the other distribution curves show obvious double peaks which have similar trends. Such a phenomenon could illustrate that results of site a and site b have no significant difference and that Bayesian estimation speed cannot efficiently decrease the distinction between instantaneous speed and average running speed under the condition of site a and site b.

In order to compare the outcomes of different installation sites more roundly, different volumes were also considered. Four levels of volumes were defined: free flow (500 pcu in arterial roads, 300 pcu in collector roads), light traffic flow (800

pcu in arterial roads, 500 pcu in collector roads), middle traffic flow (1200 pcu in arterial roads, 600 pcu in collector roads) and heavy traffic flow (1500 pcu in arterial roads, 800 pcu in collector roads). The AREs of different volumes were showed in table 2.

Table 2.The AREs of Different Volumes.

Volume		Free Flow	Light Traffic	Middle Traffic	Heavy Traffic
Collected instantaneous speed	Site a	0.070	0.062	0.058	0.059
	Site b	0.070	0.059	0.054	0.054
	Site c	0.408	0.357	0.345	0.342
Bayesian Estimation speed	Site a	0.071	0.063	0.058	0.059
	Site b	0.071	0.060	0.055	0.054
	Site c	0.154	0.135	0.129	0.114

It is clearly found out that the regularities of different installation sites and speed patterns mentioned before are satisfied under conditions of different volumes. Hence, site b could be considered to be a better installation site to choose.

Moreover, the AREs show in table 2 expressanother regularity, that is the AREs decrease along with the increasing of traffic volumes. However, the range of the decrease reduces which can be demonstrated by comparison of AREs in the same row.

4. CONCLUSION

In this paper, link travel time estimation were considered to seek the superior installation sites of loop detectors. Considering the AREs analysis and results of different traffic flows, site b-the middle of a link- have been demonstrated to be a better installation site of loop detectors since its relative lower AREs. Besides, another two regularities were also showed from the results of AREs. At first, vehicles’ instantaneous speeds at site a and site b are closer to the average running speeds than at site c, while Bayesian estimation aiming at instantaneous speeds could diminish the difference to the corresponding average running speeds. At second, a higher traffic flow causes a more accurate estimation results of link travel time, but the improvement could not be remarkable after traffic flow reach a heavier volume.

Some problems still need to be paid attention to in the future research. The major one is the influence from length of roads on estimation results. Different length of long roads were built in simulation, potential regularity may exist according to the results of site a and site b from table 1 since the shortest roads have the largest AREs. Besides, other estimation methods of link travel time could be further considered. Bayesian estimation could improve the accuracy of results from site c, other advanced or targeted estimation method may conquer the

limitation of installation sites to achieve a better result.

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Walking System Problems Analysis and Plan Conception in Chongqing City

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ABSTRACT

Walking is the main way to travel short distances, and is also one of the terminal connected to a variety of transport modes. Today, pedestrian traffic is a way to encourage, and walking quality has become an important indicator of quality of life. To many mountain cities, walking has a special meaning. People have become accustomed to walking down the street, shopping, playing, and socializing; walking has become a way of life. With the trip character in Chongqing city, this study analyzed the problems of walking system and trip behavior and proposed the requirements to the pedestrian facilities. Combined with urban waterfront space and spatial characteristics of the mountain, walking facilities are divided into three types: transportation, recreation and fitness, business. This study also proposed a different walking function system plan concept and planned a 'Mountain River City' feature walking system.

Key Words: Chongqing City; walking system; plan

1 ANALYSIS OF CHONGQING URBAN TERRAIN FEATURE

The mountain is a city, and the city is a mountain city. This is a true portrayal of geographic features unique to Chongqing's history. At the end of the Qing Dynasty Mingchen Zhang Zhidong once sang: "Chongqing city Wei Ju rock, look at sth fiercely male three bar." The status of Chongqing in the region between the main building in Liangshan and Tongluoshan, Jialing River and the Yangtze River flows through the valley, plateaus, hills. City built close to the mountains, uneven road, building well-proportioned, so it is also called "mountain city", is China's largest, the most famous mountain. A case study of Yuzhong Peninsula, which is a typical hilly topography, platform, the whole peninsula is a ridge, Chaotianmen 168 meters above sea level, with an average altitude of 249 meters of Monument for Liberation area, Loquat Hill 340 meters above sea level, the goose ridge at an altitude of about 400 meters, and the gap is in the 9 square kilometers of the Yuzhong Peninsula, you can imagine how many-storied buildings, city of ups and downs, the road is how slew, mountain is so special. In such terrain, walking is one of the people since ancient times, the main travel mode according to the

survey, over the years the city walk accounted for the main mode of transportation 50%.

Walking is one of the main way to travel a short distance, but also a variety of modes of transportation terminal connection, pedestrian traffic become encouraged development of transportation, pedestrian movement has become a lot of people especially a fitness the elderly, city walking quality has become an important index to reflect the life quality of urban residents in the city. For the mountainous city walk has special meaning, people have the habit of wandering about in the street, shopping, playing, and other friends, walking is a daily life indispensable residents of mountainous city.

In Chongqing, the special terrain, walking system is mainly divided into two categories, one is the walk in life channel, including the road on both sides of the sidewalks, pedestrian underpasses, etc., which mainly depend on road or road plaza; another is the Chongqing special terrain lead to difference, specially built for the convenience of the people travel pedestrian, the trail was initially traffic function, and development up to now is gradually transformed into leisure fitness function, the former depends on the roadway, or on the Road Plaza, which is independent of the roadway, formed a special style in Chongqing city.

2 PRESENT SITUATION AND PROBLEMS OF THE PLANNING OF PEDESTRIAN FACILITIES

According to the pedestrian facilities, will walk into traffic facilities, fitness and leisure, commercial three types:

Traffic walk -- located in the road on both sides of the sidewalk, city entrance plaza (station, dock Traffic Square), transportation (road, crossing), upper and lower transport connections (connection terminals on the stairway, tall stairway) walking road area.

Leisure fitness walking road -- located in the park (Park pedestrian system), the main public green space, city living along the mountain, along the river edge zone (Riverfront trail) and other areas of promenade.

Commercial walk -- usually in the city commercial center or group business center zone of the pedestrian street (pedestrian street, the underground pedestrian street).

2.1 Traffic walk

Yuzhong area is the largest urban area terrain elevation, is the birthplace of old Chongqing. Because of topography, contact the peninsula except several Limited Road, mostly walking streets. For a long time, people had special feelings for POE Poe kankam and lanes.

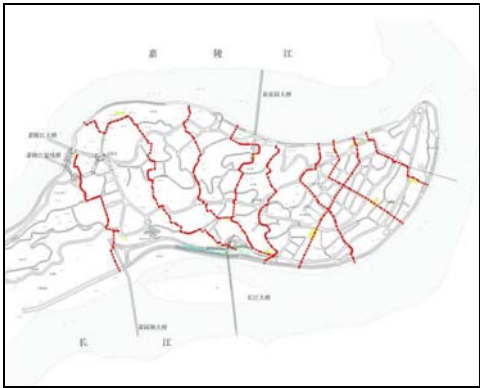


Figure 1. Schematic Diagram of Pedestrian System in Yuzhong Peninsula

Combined with the peninsula area unique topography, pedestrian channel tap and make full use of the present, a total of Yuzhong Peninsula in the nine foot path, the Southwest has eight, south-east towards a. After the construction of the continuation and years of history, the first trail (Hong Yadong -- all through the East Gate), trails and sidewalks combination method, has the guiding signs, about 1.2 km; third trails (Da Xi Gou water -- Shibampo ancient city wall), the line except the Daxigou beyond has opened, to transform the building the trail, with guide logo, is about 2.8 kilometers long. The sidewalk, the original stairway and local building transformation using the trail had connectivity, to surrounding residents living service, but the overall comfort is poor. In general, Yuzhong district has basically formed a perfect structure, unique function of walking system, constitute an important means of transportation for residents to work peninsula area travel, traffic is formed complete system structure, the continuous through, across the peninsula. Its main features are as follows:

(1) in the region has formed a vertical and main road direction, regional north-south pedestrian system traffic, and traffic is complement each other, make up the peninsula in the north and south direction because of the lack of terrain structure of road lack.

(2) the pedestrian system will be public space and focus on building the Yuzhong Peninsula has the attractive linked with balcony, corridor and green city contact semi city pedestrian stairway.

(3) cross the street area formed a relatively perfect underground pedestrian system, and solves the conflict between pedestrian and vehicular traffic, also have certain connection with the orbital station, forming a track underground pedestrian street and cross street, one of the central business district of walking system.

At the same time, there are also many problems, mainly in the:

(1) The status of building and other reasons, pedestrian channel is narrow, pedestrian comfort. Walking system has been built, width in 2~3 meters, but because of taking sides shops or street vendors, the actual available width over 1.5

meters, in time to walk more crowded pedestrian passage, next was inconvenient, many trails and steep slopes, especially for the elderly to walk is poor, and the safety pedestrian passage, in the construction of no better consider to Chongqing summer is hot and rainy climate, without considering the rain stop Yin facilities, walking comfort poor.

(2) Walk channel and traffic facilities connection and overall consideration is not enough. Pedestrian system and not a very good and bus parking port, railway station and other organic unifies in together, walk to the bus stop in Hong Kong residents or track Lane, around the larger, which had a negative effect on the residents to use public transport from the psychological.

(3) The lack of identification of pedestrian crossing, walk without a sense of direction.

(4) The pedestrian passageway in the detail aspect (green, lighting, and other pieces) design and construction deficiencies, such as the flower beds, seats placed lack, remains to be perfect.

(5) The hygienic conditions in most areas, affecting the environment and city image.

2.2 Leisure fitness walking road

2.2.1 Waterfront space

Leisure fitness trail is the characteristics of Chongqing city on the waterfront of the extending from the community, mainly located in the leisure space and outdoor leisure space, namely the park (Park Plaza pedestrian system), such as city along the mountain along the river edge (Riverfront trail) outdoor leisure spacewalk.

Chongqing is located in the upper reaches of the Yangtze River, the city built Linjiang City Mountain. The Binjiang area is a characteristic, type of landform Canyon steep slope, gentle slope, water drop is bigger, the area along the river is the natural slope, environment generally poor, some areas of rock landslide is serious, complex terrain conditions, relatively few available resources in Binjiang, and the plains of Binjiang zone has obvious difference. So the early Binjiang road construction is the comprehensive treatment project of a river.

Open the first of its kind in 1988 to 1997 in Yuzhong District, built road and Jia bin Lu, and then the main city district have built sand road, North Road, long road, South Road, Binjiang Road, so far, has completed 7 a total of about 45 km of Binjiang road in the main urban area. With the Binjiang road was built, the waterfront is the overall development and utilization.

Binjiang road in improving the city traffic, environment, landscape achieved significant results, including: improving the road network layout in Yuzhong Peninsula 1, long road, Jia bin Lu construction, to some extent alleviate traffic congestion. Mainly with the highway traffic, pier construction, dangerous rock treatment and flood control function; 2, long road with Binjiang Park Road, North Road, South Road, a small amount of hydrophilic viewing platform

facilities, increase the residents along the river hydrophilic space; land development combined with the 3, South Road, North Road, to increase the number of commercial, leisure the respect such as the service function, promote the development of the economy along the Yangtze river.

The huge social, economic benefits are very obvious to the Binjiang road. From the city since the 90's Binjiang area road construction characteristics, can clearly see the change trend of Binjiang public space environment obviously, deeper level reflects the social life style, concept, city spatial structure change tendency. In general Binjiang urban road planning and construction of the function evolution can be divided into the following three generation:

The first generation of the Binjiang road to Binjiang road Yuzhong District -- two typical, is mainly to meet the transportation, flood, landslide, environmental remediation, remediation and other functions of wharf.

The second generation of Binjiang road -- sand road, North Binjiang Road, is a typical part, based on satisfying the transportation, flood control function, in this period, greening, landscape and other aspects of the public activity space gradually taken into account.

The third generation of Binjiang road -- to riverside river Nanbin Lu and as a typical example, based on the previous two generations of Binjiang Road, pay more attention to the landscape, hydrophilic space, function, Binjiang road and surrounding areas form function, space permeability, the large scale introduction of multi-functional public activity area, but on the whole, still cannot satisfy the hydrophilic demand.

The urban area within the scope of the river Binjiang road construction speed is very fast, Binjiang road construction to improve the road network structure, promote the river waterfront land development has played a very good role. The only function of flood control of the first generation of Binjiang road to the first Nanbinbei waterfront leisure space of the third generation with the Binjiang Road, the construction mode and spatial layout waterfront are in continuous improvement, also gradually reflect the people for the casual way of life and the pursuit of a more close to nature, but due to various reasons, there are also a lot of problems, mainly in the:

(1) The early construction of the Binjiang road to only consider the function of flood control, the use of elevated and upright wall form, does not take into account the people's leisure viewing needs, making Binjiang road become hydrophilic barrier, has limited the development of waterfront space and use. Such as: Yuzhong Peninsula in Binjiang area.

(2) Binjiang with overall hydrophilic public space is not considered, especially the Binjiang area for some leisure type.

(3) Binjiang waterfront space main development by point cannot form a continuous Binjiang waterfront space with.

(4) Binjiang space walking, running poor accessibility reduces the use of function space in Binjiang.

(5) The details of the design space of Binjiang's poor, safety facilities is not fully considered. Such as: Chaotianmen, North Road, the trail.

(6) The construction of the Binjiang road traffic function demands too much attention. The construction of Binjiang road traffic is mainly in order to meet the functional requirements, less cohesive new obstacles Road, and surrounding roads, few vehicles, resulting in faster speed. Strengthen the traffic function contributes to weakening line function, pedestrian safety, noise of the Binjiang promenade of large, poor comfort, people don't want to stay here, the more impossible in sightseeing and leisure.

Overall, the area has been built Binjiang space problems can be summed up in one sentence: "heavy traffic and flood control, light landscape and environment, lack of hydrophilic space". At present, many built in Binjiang is the main road "car based", play in road traffic function, the trail is very narrow, and the system is not too much. And the construction of levees, viaduct road Shixiu, and let Binjiang road "hydrophilic space" has been destroyed. Therefore, Binjiang road in the new period built so as to a certain extent traffic diversion, but also can protect the ecological environment, the consideration of environment landscape effect, improve the pedestrian traffic system of Binjiang Road, and then increase the citizens of hydrophilic space.

2.2.2 The space

Chongqing is a river around the mountain, four separate mountain city. From east to west, the causeway mountain, in the mountain of Liangshan and Jinyun Mountain, the four main lobe formation, and Loquat Hill Park, Eling Park, Pingdingshan Park, Sha Ping Park City mountain green in the formation of a natural oxygen bar. The ancient merchants of the mountain along with society's development, has lost the original function, in the quiet gradually; and constantly improve the people's living standard, social pressure is more and more big, the demand for leisure and fitness is also enhanced, and close to nature to these natural oxygen bar leisure tourism has become the best choice for urban people's weekend holiday relaxed.

Re development of the original road form for people to leisure and fitness trails, meet the demand of people close to nature, leisure and fitness provides space for people, but also active merchants of the original, more conducive to the development and construction of mountain scenic area. But there are some problems in the early development of mountain trails.

- (1) The development and construction for the people climbing trails less.
- (2) Trails the form is single, the original stone mountain trails.
- (3) The trail corresponding facilities are not complete. Such as: lack of seat for people to rest of the way.
- (4) The trail the lack of safety facilities. Such as the guardrail, lighting and other facilities.
- (5) Without sufficient development is used for the area of the mountain park. Such as Pingdingshan park.

(6) The development and construction of the trail is still in its early stage, and mainly in the larger the space, development with the surrounding areas has not formed the system, the overall poor accessibility.

2.3 Commercial pedestrian street

At present, the main city commercial pedestrian street in the city center and four sub centers, Monument for Liberation, Guanyin Bridge, Shapingba, Nanping, Yangjiaping, is currently the five values, using loop way traffic organization, namely the garage is organized according to the single cycle mode of transportation, the main purpose of doing so is to loop is formed inside the enclosed pedestrian street, conducive to business development, its drawbacks are obvious, ring road and pedestrian inconvenience, bus transfer inconvenience, including the main problems:

(1) The pedestrian street and pedestrian scarce, pedestrian travel inconvenience.

(2) For the use of channel has been ineffective, failed to form a system, people still travel inconvenience.

(3) Green, seat, pavement and other details remain to be improved pedestrian street.

3 PLANNING IDEA WALKING DOWNTOWN

Mountain city because of the special geographical environment, limit of non-motorized travel, people generally choose pedestrian and vehicle two travel mode, travel mode has two yuan of special characteristics, at the same time travel distance, travel habits, travel psychology and trip mode has its own characteristics, people's travel demand also has the difference analysis of these characteristics, it will provide the basis for the study of pedestrian space system. Through the investigation, the Pacers have the following characteristics of city:

(1) distance feature: the Pacers because of psychological and physiological constraints, to traffic for the purpose of walking distance of no more than 5 km, preferably not more than half an hour; and by walking to convert other transport distance generally not more than 300 meters, and more than 500 meters of psychological distance was too far.

(2) the line features: the Pacers especially in traffic for the purpose of the traveler, generally through direct, fast walking route to reach the destination. Walking in the channel to transport function based, should be more considered to reduce pedestrian detour distance, reduce the Pacers in travel distance tired. At the same time, the slope must be suitable for walking on foot.

(3) environmental characteristics: Walker has a very complex and diverse needs for the walking environment. Most of the pedestrian wants to walk environment is diverse, has transformed, expected pedestrian environment have different pavement materials and pavement condition, both sides of a variety of

architectural styles to form. In the aspect of pavement material and pavement conditions, gravel, sand, gravel and the uneven ground in most cases is not appropriate, avoid wet, slippery surface, smooth road as far as possible, do not appear Gaochai, if walking channel must be ups and downs, appropriate chooses Po Road rather than step combined with the climate conditions, walking system combined with the construction, provides the rain shade conditions for the Pacers, provides a comfortable walking environment for pedestrians.

(4) features: a combination of the walking characteristics, should be in place layout, greening, seat small square, sculpture, fountain, etc..

In general, the traffic for the purpose of pedestrian travel, travel time is not more than 30 minutes, travel distance in 3000 meters, due to the main channel, should be convenient, smooth, reduce bypass. Leisure fitness for purpose of pedestrian travel, travel time should not exceed 2 hours, trip distance should be in within 10 km, walking environment to attract people to stay, landscape and pleasant, seat, greening, sketches and other facilities should be more suitable for people to stay.

4 PEDESTRIAN PLANNING SYSTEM

Combined with the Chongqing city "mountain city", "Jiangcheng" features of the city, pedestrian area, ecological park and tourist attractions important support within the city, the mountain on the potential to build urban pedestrian system, the formation of functional landscape set, in the unique tourism, leisure and fitness as well as a body of the walking space, rich residents travel mode, to create "green traffic".

4.1 Planning principles

According to the walking function and form system of the main urban areas, fully integrated pedestrian characteristics and the actual situation of city development, the pedestrian system planning principles are as follows:

(1) The overall system principle

With various regional infiltration and the city's comprehensive development of walking system, the planning and design should be included in the integration of city public space, building, space environment and the pedestrian corridor should merge into an organic whole, all elements must comply with the overall design features and tone, and should be clear, so that the whole system in order, harmony and unity.

(2) The principle of suit one's measures to local conditions

Walking system fully the unique characteristics of the combination of mountain, water, mountains, suit one's measures to local conditions, make full use of the natural environment and ecological conditions, conform to the topography, the artificial environment and the natural scenery of com., distinct personality embodies unique Chongqing landscape city.

(3) The integration of multiple functions of principle

Walking system should be a complex network system, including sidewalks, walk street, square, park, pedestrian bridges, escalators, building within the corridor, rail station etc.



Figure 2. Corridor Planning Walking Downtown

(4) Full attention and cohesion of traffic system

The full attention of cohesion pedestrian system and bus station, railway station, parking and other traffic facilities planning, fully embody the characteristics of both transportation and leisure walking system.

4.2 Pedestrian planning system

The walking system is divided into a pedestrian corridor, pedestrian street and walk three hierarchical unit.



Figure 3. District Planning Idea of Walking Downtown

Pedestrian corridor is in a certain area long distance, strong continuity, to separate pedestrian crossing or pedestrian area as the main body of the walking system, is a resident of the region surrounding the main channel to pedestrian

traffic model, as well as the focus area residents fitness, leisure and other functions in one of the. Pedestrian corridor should have strong structural and functional, can ensure the continuous, safe, comfortable pedestrian walk through the pedestrian system, walking time not less than an hour.

Walking street is in a certain area due to the large commercial facilities layout, attracted people flow specific regions more concentrated, the region has a strong human needs, has more concentrated, intensive, diversified requirements for pedestrian system area. Pedestrian multi refers to the commercial city commercial pedestrian district as the center, an area of about 3~8 square meters.



Figure 4. Unit Plan of Walking Downtown

Walking unit mainly refers to a variety of traffic aggregation, peak hour has a strong flow, in order to meet the traffic needs to walk region as the main body. Walking unit mainly concentrated in the transportation hub areas, in the area of the transport facilities as the center is about 1 square kilometers.

5 CONCLUSION

The modern traffic system, the primary way of walking is still residents, combined with Chongqing "the mountain city of Jiangcheng" characteristics, more and more people will walk as one of the main ways of leisure and fitness, with different functions, constructing pedestrian corridor, pedestrian and walking unit, to provide a good environment for the public to walk. At the same time, should pay more attention to the details of the design, construction to attract people to stay pedestrian environment.

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Scale and Distribution of Coach Stations Based on Passenger Allocation - a Case Study of Nantong City

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ABSTRACT

Nantong city will become an important transport node when the railway network perfects in 2020. Then coaches from Nantong city will serve new railway transfer passengers, increasing regional and local passengers. Both types and the destination proportion will change obviously. In the research we allocate the passengers into current and preplanned coach stations with the principles of accessibility, least transportation interference and convenience. Four schemes are compared to determine the best scale and distribution of coach stations for better services and operating efficiency.

Key words Coach station; Scale; Distribution; Passenger allocation; Nantong city

1 BACKGROUND AND RESEARCH SIGNIFICANCE

1.1 Background

Nantong City is on the opposite Yangtze River bank of Shanghai (Figure 1). It finished the water-land transshipment with Shanghai in 2008 after the construction of Sutong Bridge. Meanwhile highway transportation boosts tremendously and passenger flow witnessed a great increase to 98.2% in 2012. With Hutong (Nantong to Shanghai), Tongsujia (Nantong to Jiaxing) and Ningqi (Nantong to Nanjing and Qidong) three newly built railway lines in 2020, Nantong city will become a node in the regional transportation network from an end point nowadays.



Figure 1. Location of Nantong City

Currently, passengers from Nantong go out by coach mainly. The new railway lines will change it soon, just as many other cities in recent years. When the integrated railway network perfects, there will be a significant expand of railway market share and coaches will play a key role of short shuttles.

1.2 Research significance

In 2020, the railway and highway passenger rate will change from current 1.2:98.2 (water carriage owns the other 0.6) to 12:88 (Nantong Railway Bureau, 2009). On one hand, coaches will serve new railway transfer passengers; on the other hand, the coach market grows quickly with urbanization progress. So, current two coach stations in Nantong city will no longer meet the increasing demands. The research on scale and distribution of coach stations can determine a better station plan and help them providing better services and improving operating efficiency.

2 LITERATURE REVIEW

The study on the scale of transport facilities focused on travel demand forecasting model. In 1955, Chicago area transportation study used a four-stage method. Ruitter and Ben-Akiva (1978) built the first econometrics model (MTC system) to forecast. In China, some updating models and methods were put forward in recent years. Liu (2002) proposed five factors model to calculate the transit hub scale and quite a lot

researches implemented multi-variable models to calculate (Huang, 2004; Li, 2009; Wang and Ding, 2011).

Site selection theory includes static state and dynamic state methods. The latter is always seen as more objective. Hostelling (1929) led tools of game theory into analysis, Smithies (1941) formed the factors of demand and transportation costs without considering urban network. Current et al. (1997) calculated the minimization of expected opportunity cost and loss. Heuristic approaches (Da Gama and Captivo, 1998; Wanger, 2008) and model based on necessary factors (Gromule and Yatskiv, 2007) are introduced to China to be used in practice. White (2012) analysed the market of coach stations.

Nowadays some commonly used methods for scale forecasting and distribution are used in practice in Chinese cities such as Tianjin, Dalian, Urumqi, Shijiazhuang, Shenyang, Xi'an, Yingkou, Chifeng, Songyuan and so on. Most of them focus on single station and study single passengers' source.

3 RESEARCH DESIGN

This research will make a breakthrough to consider regional transportation infrastructure construction as an important factor affecting coach station scale and to take account of the transfer flow between trains and coaches. The research is based on the operating reports of Nantong Transportation Bureau and the surveys of Nantong Coach Station and Nantong North Railway Station in January 2013. We collected data of passengers' origins and destinations and the transfer flow between trains and coaches. The coach routes from the research are divided into regional routes (cities outside Nantong) and local shuttles (county-level cities in Nantong). Regional routes might change drastically due to the influence of new railways because trains are much more comfortable, safer and faster. Meanwhile, local shuttles would be much steadier because of flexibility.

So, coach passengers from Nantong city were divided into three parts (Figure 2): railway transfer passengers to county-level cities, local shuttle passengers to county-level cities, regional passengers to outside cities. A small amount of coach transfer will be divided to part 2 or part 3 as two journeys.

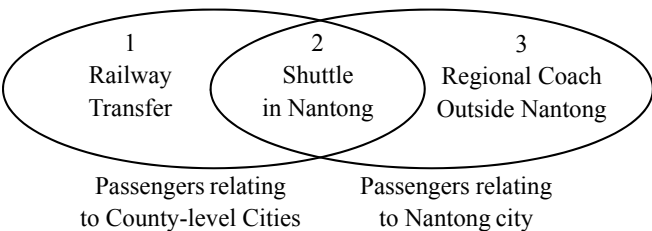


Figure 2. Composition of Coach Passengers from Nantong City

Firstly, we study the accessibility of coach stations linked with railway stations by comparing them with the total time cost of county-level cities. It comes from the multiplication of passenger number and time that comes from the timelines through freeway and highway in GIS software. Railway transfer passengers will be allocated on the principle of accessibility. Secondly, regional routes will be changed to set off from certain stations have the same orientation with destinations so there is least interference to urban internal transportation. According to destinations, regional passengers will be allocated to different stations. Thirdly, local shuttle passengers will be allocated on both principles of accessibility and least interference. But it is different from regional routes because more than one station will have the same destination based on convenience. Lastly, the three parts of passengers will be summed and the best scale and distribution should be determined under the necessary conditions.

4 STATION CONDITIONS AND DISTRIBUTION SCHEMES

4.1 Master plan of Nantong

According to the Master Plan of Nantong (2011-2020), the mainly built up area concentrate on three districts: Gangzha with a planned population of 450000, Chongchuan of 900000 and Economic Development of 450000. We draw the the border of planned built up area approximately in figure 3. Some eastern parts of the area belongs to Tongzhou district but not adjoined to Tongzhou city. So we make Tongzhou as a single county-level city in subsequent data analysis.

The other five county-level cities (excluding Tongzhou) in Nantong domain are Hai'an, Rugao, Rudong, Haimen and Qidong (Figure 1).

4.2 Coach Stations

There are two current and three preplanned coach stations in master plan and traffic plans (Figure 3): Nantong Station is the oldest which located in downtown area with best service scope but interferes with internal traffic of the city. East Station is newly constructed partly for Tongzhou County-level city (it has been adopted as one district of Nantong in 2009) that is close to Nantong. North station is just a small stop now, and it is planned to serve for Ningqi Railway Station in the future but its service scope overlaps with Nantong station. The planned West Station seamlessly connects Hutong (Tongsujia) Railway Station but is quite far from built up area. The other planned Development District Station will serve in the southern industrial area and only for certain local shuttles.

4.3 Station distribution schemes

According to current conditions and plans, four schemes are made: Scheme1 is to build West Transit Center including railway and coach stations and cancel North Station. Scheme2 is to build North Transit Center and cancel West Station. Scheme3 is to close Nantong Station and construct West and North Transit Center. Scheme4 is to keep all stations and transit centers operating.



Figure 3. Locations of Coach Stations in Nantong City

5 SCALE CALCULATION OF COACH STATIONS

5.1 Railway transfer passengers to county-level cities

There are average 23000 passengers starting from Nantong city by coach everyday (Nantong Transportation Bureau, 2012). The survey indicated that 3% of them transferring from railway. Calculating on the 12th five-year plan of railway and highway constructions in Nantong, the passengers number will reach to 48000 and the proportion will increase to 9.7% in 2020.

Based on survey and plan we infer the proportion of six county-level cities and calculate the travel time from West and North Station by GIS. So we get the passenger time cost (Table 1).

Table 1. Passengers Time Cost From West and North Station To County-Level Cities

Destination	Proportion	Passengers number	Travel time from West Station (hour)	Passenger time cost (p*h)	Travel time from North Station (hour)	Passenger time cost (p*h)
Tongzhou	0.32%	153	0.67	103	0.57	88
Rugao	2.46%	1181	0.57	673	0.67	791
Qidong	3.06%	1469	0.83	1219	0.67	984
Haimen	0.81%	389	0.67	260	0.67	260
Haian	1.34%	643	0.83	534	0.83	534
Rudong	1.71%	821	1.5	1231	1.5	1231
Total	9.7%	4656		4020		3888

Two stations differ little. Hence, according to the railway trains plan, transferring passengers were allocated in terms of railway passenger proportion of two railway stations which is 55:45. So West and North Station were planned to send 2561 and 2095 transfer passengers every day respectively in scheme3 and 4.

5.2 Regional passengers to outside cities

Regional routes connecting southern cities are arranged in East Station and north in North Station, west in West Station or Nantong Station.

Current regional passengers from Nantong city is 60% in total passengers (the total number is excluded the very few railway transfer passengers). It will be about 70% estimated in 2020 for rapid increasing population scale in Nantong urbanization progress. So the proportion is adjusted as Table 2 on the basis of railway market share in 2020. The allocation of passengers in each station of four schemes can be seen in Table 3.

Table 2. Planned Regional Coach Passenger Scale

Destinations	Current proportion	Adjusted proportion	Passenger number
Nanjing, Yangzhou, Taizhou	14.85%	5.27%	1600
Suzhou	23.03%	8.18%	2483
Shanghai	16.72%	5.94%	1802
Wuxi, Changzhou, Zhenjiang	18.40%	32.67%	9913
Other cities in north Jiangsu Province	13.14%	23.33%	7078
Cities in Zhejiang Province	6.83%	12.12%	3678
Cities in other Province	7.04%	12.49%	3788
total	100%	100%	30342

Table 3. Regional Passengers' Allocation of Four Schemes

	W.	N.	E.	Nantong	W.	N.	E.	Nantong
Destinations	Scheme1				Scheme2			
Nanjing, Yangzhou, Taizhou	0	-	0	1600	-	0	0	1600
Suzhou	0	-	2483	0	-	0	2483	0
Shanghai	0	-	1802	0	-	0	1802	0
Wuxi, Changzhou, Zhenjiang	0	-	0	9913	-	0	0	9913
Other cities in north Jiangsu Province	7078	-	0	0	-	7078	0	0
Cities in Zhejiang Province	0	-	3678	0	-	0	3678	0
Cities in other Province	0	-	3788	0	-	0	3788	0
Total	7078	-	11751	11513	-	7078	11751	11513
Destinations	Scheme3				Scheme4			
Nanjing, Yangzhou, Taizhou	1600	0	0	-	1600	0	0	0
Suzhou	0	0	2483	-	0	0	2483	0
Shanghai	0	0	1802	-	0	0	1802	0
Wuxi, Changzhou, Zhenjiang	0	9913	0	-	0	0	0	9913
Other cities in north Jiangsu Province	0	7078	0	-	0	7078	0	0
Cities in Zhejiang Province	0	0	3678	-	0	0	3678	0
Cities in other Province	0	0	3788	-	0	0	3788	0
Total	1600	16991	11751	-	1600	7078	11751	9913

It can be seen that when West or North Station is cancelled in scheme1 or 2, Nantong Station and East Station will take charge of most of the passengers. It is because only the routes to north Jiangsu Province starting from West or North Station. Once Nantong Station is cancelled in scheme3, North Station's scale increases significantly. Because the former passengers of Nantong Station are mostly moved to North Station and meanwhile East Station maintains the same scale for the changeless routes. West Station has a small scale because only west-destination routes start from here. If all the stations are kept operating in scheme4, Nantong Station's scale will become much smaller.

5.3 Local shuttle passengers to county-level cities

According to the traffic plan, the shuttle routes from Nantong city to small towns in the counties will be cancelled in 2020. The station will take these passengers to the county-level cities to transfer.

Local shuttle passengers from Nantong city is 30% in total passengers (excluded railway transfer). So, based on current proportion of each county-level cities' passengers and railway plan, we estimate the new proportion and allocate the planned passengers in four schemes (Table 4).

It can be seen that East Station has the largest scale of shuttles. Because the east area in Nantong is more developed and has more connections with Nantong city. By contrast, the north area has fewer connections with Nantong city.

Table 4. Local Shuttle Passengers’ Allocation of Four Schemes

	Tongzhou	Haimen, Qidong	Rudong	Rugao, Hai'an	Total
stations	Scheme1				
West	0	780	260	910	1950
North	0	0	0	0	0
East	5461	1560	910	1820	9752
Nantong	0	0	0	0	0
Development	607	260	130	303	1300
stations	Scheme2				
West	0	0	0	0	0
North	0	780	260	910	1950
East	5461	1560	910	1820	9752
Nantong	0	0	0	0	0
Development	607	260	130	303	1300
stations	Scheme3				
West	0	390	0	0	390
North	0	390	260	910	1560
East	5461	1560	910	1820	9752
Nantong	0	0	0	0	0
Development	607	260	130	303	1300
stations	Scheme4				
West	0	390	0	0	390
North	0	390	260	910	1560
East	5461	1560	910	1820	9752
Nantong	0	0	0	0	0
Development	607	260	130	303	1300

6 CONCLUSION

The passengers of three parts are summed respectively in Table 5.

Table 5. Coach Station Scale of Four Schemes

	Scheme1		Scheme2		Scheme3		Scheme4	
Stations	Passenger number	Area (ha)	Passenger number	Area (ha)	Passenger number	Area (ha)	Passenger number	Area (ha)
West	13684	6	0	0	4641	3	4641	3
North	0	0	13684	6	20646	9	10733	6
East	21503	9.3	21503	9.3	21503	9.3	21503	9.3
Nantong	11513	5	11513	5	0	0	9913	5
Development	1300	6	1300	6	1300	6	1300	6

Scheme 1, 2, 4 will have three first-level coach stations and two stations' scale is not much more than 10000 passengers. As 10000 passengers is the lowest level for first-level coach station, the construction isn't intensive. Scheme3 has two first-level coach stations and both of them larger than 20000. So the passenger flow will be concentrated and investments can be saved. On the other hand, two stations in scheme3 locate Gangzha and Chongchuan Districts respectively, so they have good service scopes and they are convenient to transfer between regional coaches and shuttles in the same station.

As a first-level coach station, West Station in scheme1 is out of built up area and over 13000 passengers have to spend a lot of time on the way. In scheme1, 2 and 4, Nantong Station's daily 10000 passengers will add burden to the bad urban traffic. Scheme3 will transform Nantong Station's function to tourism and it is helpful to traffic organization.

In sum, scheme 3 is the best for Nantong.

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A Common Methodology for ITS Benefits Calculation of GHG

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ABSTRACT

This paper describes the role of ITS in implementing California's Global Warming Solutions Act of 2006, which requires reduction of GHG emissions to 1990 levels by 2020. The paper considers the "Common Methodology" being developed for use at the international level to enable independent third party verification of GHG emission reductions due to ITS. Evaluation is made of the \$7.6 billion in ITS planned in the Regional Transportation Plan and Sustainable Community Strategies (RTP/SCS) adopted by the Southern California Association of Governments (SCAG). These ITS projects are to help achieve a nine (9) percent reduction by 2020 compared to the 2005 level on a per capita basis. Analysis is made of the SCAG 2012 – 2035 RTP/SCS planned ITS for enhanced incident management, advanced ramp metering, traffic signal synchronization, advanced traveler information, improved data collection, universal transit fare cards (Smart Cards), and Transit Automatic Vehicle Location (AVL) to increase traffic flow and reduce congestion to consider whether these are the most beneficial ITS investments. Transportation policy-makers in Europe, Japan and the US are cooperatively developing a Common Methodology that can be accepted and widely used at the international level. Qualitative analysis has been drafted to rate ITS categories for potential GHG emission reductions, and case studies are planned to enable the first version of the Common Methodology to be available for use by the end of 2016.

INTRODUCTION

This paper describes the value of an internationally accepted "Common Methodology" protocol for measuring benefits of implementing ITS to help achieve California's Global Warming Solutions Act. This State law requires reduction of greenhouse gas (GHG) emissions to 1990 levels by 2020. The Common Methodology is being developed for use internationally via a cooperative effort of the European Union ECOSTAND Project, Japan's Energy ITS Project, and the United States Applications for the Environment Real Time Synthesis (AERIS) Program.

The Common Methodology is envisioned for use by agencies to provide evidence of "green credentials" and to document that targeted GHG reductions have been achieved. Furthermore the agency will be able to ask, through the Common Methodology website, to a qualified body the certification of credential

(ECOSTAND, 2012). It therefore needs a certified method for assessing the combined impact of the ITS systems deployed, with an independent third party verification of GHG emission reductions due to ITS.

The Roadmap to this methodology is depicted in Figure 1. It calls for an internationally recognized standard by 2016. It is now proceeding with definition of stakeholders, and with validation of the correctness of modelling approach for each ITS application category. These short term activities are considered in the following sections of this paper.

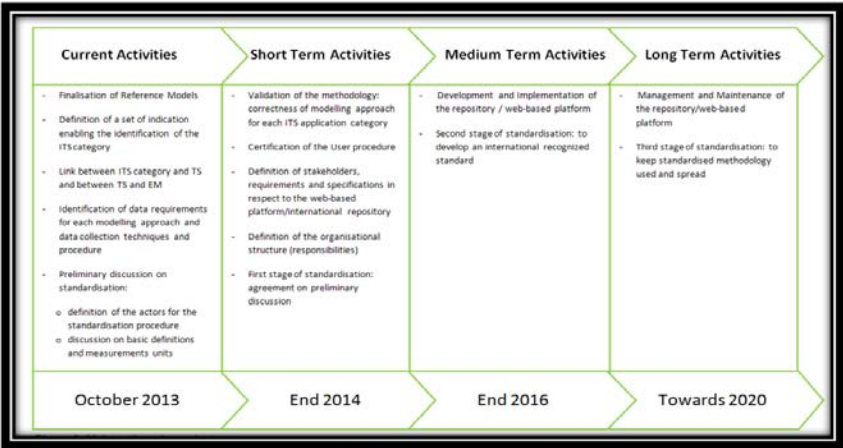


Figure 1. The Roadmap Timeline

COMMON METHODOLOGY STAKEHOLDERS

Common Methodology participants have identified that a “crucial aspect for the next phase is to extend awareness of the preliminary Methodology and Roadmap in order to gather opinions and support from a wider public...and future potential users of the Methodology.” (Spence et al., 2012)

Regions that embrace a sustainable future are the stakeholders that are getting ahead of the climate change curve – regions that are enabling a future that sets the table for businesses to create profit for its shareholders while protecting the environment and improving the lives of those with whom it interacts. These regions operate so that its business interests and the interests of the environment and society intersect.

To conform to requirements of the California Global Warming Solutions Act and related legislation the Southern California Association of Governments (SCAG), the largest Metropolitan Planning Organization in the United States, adopted the 2012-2035 Regional Transportation Plan/Sustainable Communities Strategy (RTP/SCS): Towards a Sustainable Future. ITS is incorporated for its critical role in the operation and management strategies designed to increase the efficiency of the existing transportation system.

The 2012 RTP/SCS allocates \$7.6 billion in Transportation System Management measures, including ITS for ramp metering, increased freeway detection, monitoring of goods movement operations, advanced traveler information systems/5-1-1/goods movement information systems, transit vehicle location and real-time schedule adherence, Rapid Bus systems, computer-based signal timing systems, interconnection between roadway traffic signals and railroad signal systems, automated fare collection and toll collection technologies.

These ITS investments are intended to help achieve a nine (9) percent reduction in GHG by 2020 compared to the 2005 level on a per capita basis. They are also intended to increase mobility within existing pavement widths, to the benefit of businesses, the environment and society.

To deliver on these intentions, funding must be allocated for ongoing operations and maintenance. A potential source for such funding is through the California Cap-and-Trade Program. This approach would enable businesses to be part of the solution to the world's environmental and social problems. Rather than having a strict government mandate like a carbon tax, the cap-and-trade system uses market mechanisms to reward companies that figure out ways to reduce pollution below the levels set by government. California's refineries, power plants and large factories must obtain permits, called "allowances" for each metric ton of GHG emitted by their operations. By holding electronic auctions of allowances, California will make \$2 Billion to \$14 Billion per year, depending on the demand for the allowances, which may be allocated for ITS operations and maintenance for GHG reductions as verified by the Common Methodology.

Another possible approach to achieve the "triple bottom line" where business interests and the interests of the environment and society intersect would entail the use of "Offset credits". An *offset* is a credit that represents a reduction or removal of greenhouse gases by an activity that can be measured, quantified, and verified. Individual offset projects can be implemented to generate offset credits, which can then be sold and used by a covered entity as a compliance instrument in the Cap-and-Trade Regulation. Under the California Cap-and-Trade Program, covered entities may use offset credits to satisfy up to 8 percent of their total compliance obligation for each compliance period. Offsets are tradable credits that represent verified GHG emission reductions in sectors and sources not covered under the cap. Although the offset project is not itself covered under the cap, it can generate reductions for use by entities who must comply with the Regulation. The inclusion of offsets in the program will support the development of innovative projects and technologies from sources not subject to a compliance obligation. Offset credits represents a reduction of one CO₂e GHG emissions from an approved uncapped source. An uncapped source is an entity that is not regulated by the cap-and-trade program, such as surface transportation. Credits are only awarded to GHG emission reductions carried out pursuant to Protocols promulgated the California Air Resources Board (CARB). So far, CARB has only approved GHG offset projects in four categories: Forest Projects, Urban Forest Projects, Livestock Projects, and Ozone Depleting Substance Projects. These fall far short of the potential demand as shown in Figures 2 and 3.



Figure 2. Total Offset Demand

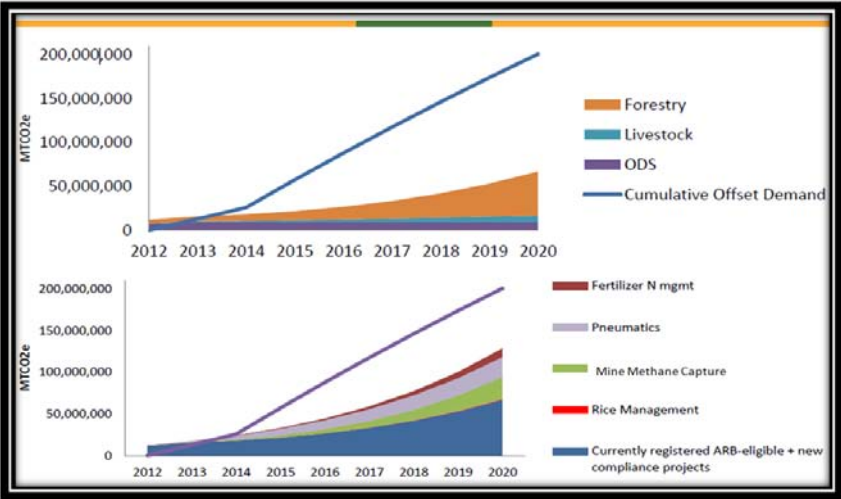


Figure 3. CARB Offset Demand vs. Potential Supply

In a 2013 court test, the California court found “Additionality is the linchpin of an offset program. A reduction is additional if it would not have occurred without the financial incentive provided by the offset credit. Additionality is essential to the environmental integrity of an offset program because if reductions are not additional,

then the cap-and-trade program will not reduce GHG emissions beyond what would have occurred anyway.”

For its four GHG offset Protocols, CARB adopted a “standard-based approach,” relying on information about the additionality of categories of prospects, rather than to evaluate each offset project’s additionality individually, project-by-project, based on site-specific data and parameters. The court recognized the important roles that GHG offsets play in reducing the cost of GHG emission reductions and promoting innovation. Such a standard based approach is consistent with the Common Methodology approach.

VALIDATION OF MODELING APPROACH FOR ITS CATEGORIES

The Common Methodology participants are progressing through the modeling of ITS applications per the approach shown in Figure 4. Five ITS categories have been identified to include 1) improving driving behavior, 2) energy-efficient traffic control for intersections and corridors; 3) energy-efficient operations on a network scale; 4) travel demand management; and 5) fleet management.

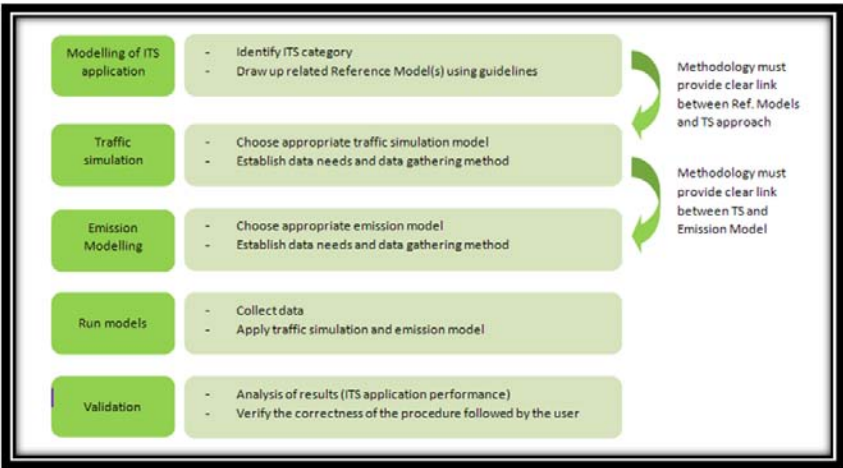


Figure 4. Common Methodology Approach

It is recognized that some Instance Models still also need to be developed to fulfill their agreed upon process. The United States is starting to develop an approach to modeling energy and emissions reductions for their environmentally-focused ITS projects (Kuwahara et al., 2013). In the U.S., there have been a number of different efforts to model the reduction of energy and emissions for ITS projects over the last two decades. These efforts have typically been part of individual projects and have lacked coordination between them. However in recent years, the U.S. Department of Transportation has established an environmentally-focused ITS research program called Applications for the Environment: Real-Time Information Synthesis or AERIS. The goals of the AERIS program focus on environmental ITS solutions by generating

and acquiring environmentally-relevant real-time transportation data and then using these data to create actionable information that support and facilitate “green” transportation choices by transportation system users and operators. This is part of the larger connected vehicle research program to illustrate how connected vehicle data and applications will contribute to mitigating the negative environmental impacts of surface transportation.

Transformative concepts are defined to consist of applications or technological solutions designed to ingest, process, and disseminate data in order to address specific tasks and combine applications that result in significant environmental benefits to surface transportation networks. Currently there are six AERIS Transformative Concepts which include: 1) Eco-Signal Operations; 2) Eco-Lanes; 3) Dynamic Low Emissions Zones; 4) Support for Alternative Fuel Vehicle (AFV) Operations; 5) Eco-Traveler Information, and 6) Eco-Integrated Corridor Management (ICM).

Table 1 relates the five categories of ITS identified with the Common Methodology with the AERIS transformative concepts and with the ITS components includes in the SCAG 2012 – 2035 RTP/SCS.

TABLE 1. Common Methodology ITS Categories

COMMON METHODOLOGY CATEGORY	AERIS Transformative Concepts	ITS Projects in SCAG RTP/SCS		
Category 1: Improving Driving Behavior	Eco-Traveler Information	Advanced Traveler Information		
		Improved Data Collection		
Category 2: Energy-efficient Traffic Control (for intersections and highway corridors)	Eco-Signal Operations	Enhanced Incident Management		
	Eco-Lanes	Advanced Ramp Metering		
		Traffic Signal Synchronization		
Category 3: Energy-efficient Traffic Management on a Network Scale	Dynamic Low Emissions Zones; Eco-Integrated Corridor Management	None		
Category 4: Travel Demand Management (ITS-related transport policies to influence travel behavior and modal choice)	Support for Alternative Fuel Vehicle Operations	Universal Transit Fare Cards		
		Transit Automatic Vehicle Location		
Category 5: Fleet Management (for commercial and public transport fleets)	Support for Alternative Fuel Vehicle Operations	None		

ITS investments planned by SCAG emphasize the Common Methodology Category 2, Energy-efficient traffic control for intersections and highway corridors, with a large investment in traffic signal synchronization. One of the more challenging problems in modeling traffic signal synchronization applications is the consideration of different elements of the trip chain: destination choice, mode choice, time of day choice, route choice and lane choice. This influence will vary from one application to another and feedback loops between behavior and traffic assignment models will have to be created to capture these effects. A modeling approach is being developed that captures the effects of AERIS applications on all levels, regional to individual intersections.

Micro simulation tools will be used to carry out a high resolution simulation of vehicles and capture their detailed trajectories for the modeled region. The

approach will be to use a variety of micro simulation tools to model key areas of activities (e.g., traffic intersections, corridors of coordinated ITS activities, links with specialized ITS elements, etc.) and then aggregate these results upward into the macro scale modeling. As such, efforts will be taken to establish the micro simulation “building blocks” and then use the results of the building blocks in the larger regional modeling.

CONCLUSION

Great progress is being made in development of a Common Methodology for international use to document the GHG-reducing benefits of ITS operations. Existing ITS projects with already developed micro simulation models will be useful in the next steps along the Common Methodology roadmap. This paper was prepared to show the potential value of the Common Methodology and to extend awareness of the preliminary Methodology and Roadmap to gather opinions and support from future potential users.

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Factors that Influence the Introduction of New-energy Buses into Taiwan

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ABSTRACT

International trends show that the vehicle industry is moving towards energy-saving, safe, comfortable and entertainment-oriented vehicles. The operating patterns and energy-saving features of new-energy buses match these trends, and will move them to the center of energy development policies. Although the central government and local governments have subsidy policies for the purchase of new-energy buses, the subsidies are apparently not enough for operators to effectively support the new-energy bus industry. This research attempts to discover the influencing factors in introducing new-energy buses to Taiwan through data analysis and interviews with industry operators and owners. The results showed that even if purchase costs are reduced with the help of subsidies, operators still would not purchase new-energy buses. This is mainly due to a lack of technology, an immature industrial environment, and insufficient government policies, as well as other factors.

1. INTRODUCTION

In response to the growing global trend of energy conservation, industries around the world have begun to adopt energy-saving technologies. In the automobile industry, specific approaches to realizing energy conservation include "alternative fuel technologies" and "advanced vehicle technologies". The development of alternative fuel technologies involves the use of fuels derived from products other than petroleum as the power sources for vehicles, such as natural gas and biofuels. Advanced vehicle technologies, on the other hand, involve the design and development of advanced propulsion equipment to replace the conventional internal combustion engine, such as electric vehicles and fuel cell vehicles. (Institute of Transportation, Ministry of Transportation and Communications, 2011). In recent years, an entirely new approach for accomplishing energy conservation has been developed: retroactively installing energy-saving technologies into conventional vehicles to enable them to achieve a level of energy efficiency comparable to that provided by advanced vehicle technologies (as reported in studies by the Bureau of Energy, Ministry of Economic Affairs in 2010). An example is the hydraulic power hybrid system. The approaches mentioned above demonstrate that the automotive industry has indeed been

following the trend of energy conservation and carbon reduction.

Apart from the clear trend that has continued on a global scale, there have been a number of recent studies on this topic carried out in Taiwan (e.g., the pilot project for operating electric buses) and government promotion policies (e.g., the ten-year diesel vehicles replacement project initiated by the MOTC), the purpose of which is to achieve attain the goal of energy saving and carbon reduction. Take electric buses as an example. In Taiwan there are currently 61 electric buses operating in Taipei City, New Taipei City, Taoyuan County, Greater Hsinchu and Kaohsiung City. Generally speaking, Taiwan's electric bus industry has now only entered the initial stage of development. As both market conditions and government subsidy policy become favorable, the utilization of electric buses is expected to take off.

2. DISCUSS OF FACTORS THAT INFLUENCE THE INTRODUCTION OF NEW-ENERGY BUSES INTO TAIWAN

Based on the above observations and assumptions, this study attempts to promote the adoption of electric buses in Taiwan through interviews with relevant industry players as well as operating cost calculations associated with the deployment of electric bus fleets. Our discussion below will be conducted based on technical, industrial environment and government policy aspects.

2.1 Technical

Entities associated with the development of electric bus technologies in Taiwan include research institutions and individual corporations. Examples of research institutions involved are: the Automotive Research & Testing Center (ARTC) and the Industrial Technology Research Institute (ITRI); Corporations from the industry include RAC Electric Vehicles, Aleees, PONY Rent-A-Car, All Win Green Power Technology, Pihsiang Energy Technology, Yulon HAITEC, and Tang Eng. The interviews conducted with these corporations indicate that there are two modes of electric bus battery utilization: Battery recharging and battery replacement. The technologies associated with these two modes of battery operations are quite mature, and they offer battery life in the range of 120 km to 300 km (when fully charged). However, the experience with operating electric buses in Taiwan is currently quite limited, and as a result the jury is still out on battery safety and stability.

2.2 Industrial Environment

As stated in the introduction, the rolling out of electric buses in Taiwan remains in its infancy, and the general conditions for their deployment are less than favorable to operators. According to data collected from the interviews, two problems currently remain: (1) government regulatory authorities have not provided

sufficient information to bus operators; (2) Battery recharging facilities are inadequate, resulting in the slow adoption of electric buses in Taiwan. On the issue of insufficient information received by bus operators, while other countries have enjoyed government-led pilot programs in promoting electric buses, authorities in Taiwan have yet to play an active role in this capacity. If a municipality wishes to develop its own electric bus demonstration program, it is likely that the project will have to be undertaken jointly by the local government and commercial operators themselves. In addition, the national government has not been involved with any commercial operations following the delivery of vehicles. With respect to the issue of inadequate recharging facilities, at the moment the electric bus transportation industry remains at the mercy of battery manufacturers, compared with the situation of refueling stations for diesel buses. Without a viable recharging infrastructure, electric buses can only be deployed as shuttles for shorter routes. Nor can the vehicles be utilized by other transportation operators as chartered buses. In other words, the scope of their use becomes very limited.

2.3 Government policy

According to the Institute of Transportation, the promotion strategies for vehicles utilizing alternative energy released in 2011 include economic incentives, traffic management, transportation industry regulation, vehicle supervision, infrastructure, and benchmark demonstration. The results of the interviews show that economic incentives and infrastructure remain the major motivations for promoting electric buses. However, at present government subsidies for investing in infrastructures are rather limited in Taiwan, and therefore this study focuses only on subsidies to the purchase of electric buses. Both the central and local governments have currently developed policies on subsidies provided to the purchase of electric buses. Subsidies granted by the central government include the following: (1) Up to NT\$5.2 million for buses used on new service routes (80% of the purchase of each vehicle, excluding batteries); (2) Up to NT\$3.85 million for the replacement of old vehicles on existing services routes (49% of the purchase price of each vehicle, excluding batteries); Subsidies from local governments will be based on individual projects and determined through application. As an example, Taoyuan County provides a subsidy of up to NT\$1.5 million per vehicle. Overall, under the current government policy on subsidies, the actual amount of investment bus operators are required to make in the acquisition of electric buses (excluding subsidies) does not differ significantly from that for purchasing conventional diesel-based vehicles. However, confidence in using this new technology remains the major issue with bus operators when making the decision to purchase electric buses.

3. PARAMETERS SET ABOUT OPERATION COST CALCULATE BY ELECTRICAL BUS

Based on the interviews conducted in this study and the relevant documents and data collected in the process, we have come up with tentative calculations on the costs of operating electric buses and the cost difference when compared with conventional diesel vehicles, which we hope will be useful to bus operators in their decision-making processes. Refer to Table 1 for the model parameters that have been established.

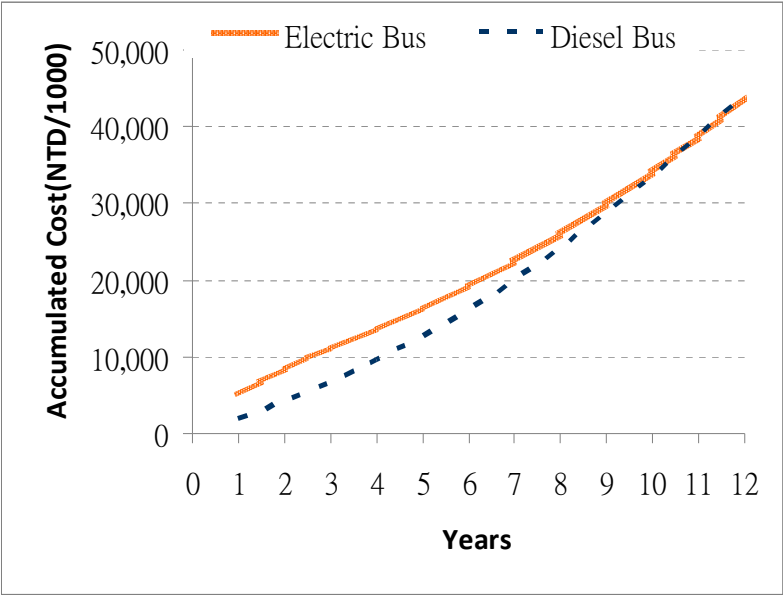
Table1. Parameter Setting of Operation Cost by Electric Bus

Category	Setting		Unit	Reference
Operator	Charge base	12	NTD per km	Interview
	Operation period	260	Day per year	The Study
	Electric Bus Value(No Battery)	8,500	Thousand	Interview
	Diesel Bus Value	5,000		
	Maintenance Fee	376		
Battery Stakeholder	Battery Life Cycle(General)	3	Year	
Government	Fuel Price	32.2	NTD per l	The Study

Source: The Study.

4. RESULTS

We have constructed a cost model in this study. Items included in estimating the operating costs of electric buses are: vehicle depreciation cost, vehicle ownership cost and energy consumption cost. In order to comply with current laws and regulations, electric buses are required to provide a service period of no less than 8 years; as conventional diesel-based vehicles have an operating life of 12 years, this study provides the tentative cost calculations based on an evaluation period of 12 years. Table 2 and Figure 1 provide the operating cost calculations of electric buses and conventional diesel vehicles based on the operating parameters listed in Table 1.



Source: The Study.

Figure 1. Difference of Operation Cost between Electric Bus and Diesel Bus (Without Subsidy)

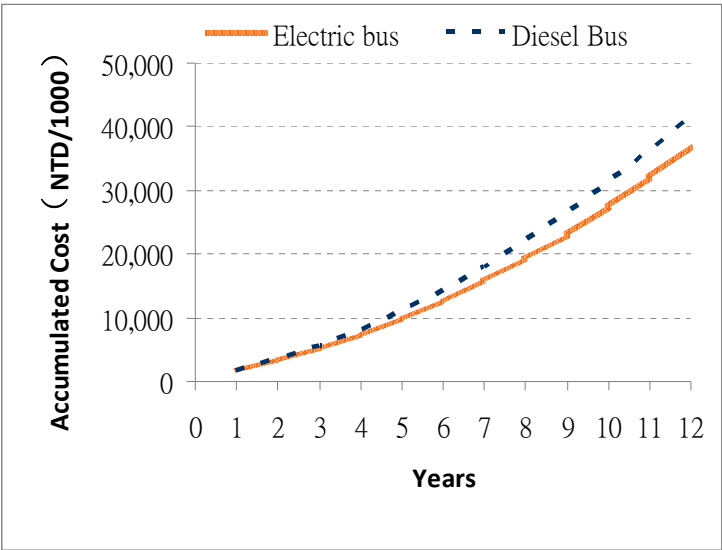
Table 2. Operation Cost Compose between Electric Bus and Diesel Bus (Without Subsidy)

Unit: Thousand

Evaluate period (Year)	Electric Bus				Diesel Bus			
	Depreciation Cost	Operation Cost	Energy Cost	Accumulation	Depreciation Cost	Operation Cost	Energy Cost	Accumulation
	(No battery)				(No battery)			
1	4,250.01	470.48	449.28	5,169.77	870.38	455.98	634.51	1,960.87
2	6,375.02	1,289.61	898.56	8,563.19	1,592.60	1,273.48	1,307.10	4,173.18
3	7,437.52	2,457.89	1,347.84	11,243.25	2,196.30	2,452.76	2,020.05	6,669.11
4	7,968.77	3,986.31	1,797.12	13,752.20	2,705.19	3,996.32	2,775.78	9,477.29
5	8,234.40	5,901.97	2,246.40	16,382.77	3,138.23	5,921.82	3,576.86	12,636.91
6	8,367.21	8,202.44	2,695.68	19,265.33	3,510.59	8,227.99	4,397.97	16,136.55
7	8,433.62	10,864.53	3,144.96	22,443.11	3,834.40	10,902.54	5,239.61	19,976.55
8	8,466.82	13,900.23	3,594.24	25,961.29	4,119.37	13,951.84	6,102.30	24,173.50
9	8,483.42	17,309.80	4,043.52	29,836.74	4,373.28	17,376.00	6,986.56	28,735.83
10	8,491.72	21,093.44	4,492.80	34,077.96	4,602.33	21,175.13	7,892.93	33,670.39
11	8,495.87	25,251.34	4,942.08	38,689.29	4,681.87	25,349.36	8,821.96	38,853.18
12	8,497.95	29,783.69	5,391.36	43,673.00	4,745.50	29,898.75	9,774.22	44,418.47

Source: The Study.

The results of this study show that—based on per kilometer energy cost calculations—bus operators will not incur substantial additional expenses, as they are not required to assume any battery-related costs. The overall trend, however, suggests slightly higher operating costs for electric buses compared with conventional diesel vehicles (government subsidies are excluded from the tentative calculations above). When government subsidies are also included, the cost differences between electric and conventional diesel buses are as shown in Figure 2.



Source: The Study.

Figure 2. Difference of Operation Cost between Electric Bus and Diesel Bus (With Subsidy)

5. CONCLUSION

These calculations indicate that the costs of operating electric buses will be slightly higher than those of conventional vehicles when government subsidies provided to the purchase of electric buses are not taken into consideration. On the other hand, when government subsidies are included, it will be less costly to operate electric buses than conventional diesel vehicles. In light of the global move toward energy conservation and carbon reduction—as well as increasingly higher oil prices—we believe that the results of this study will prove to be a particularly useful reference for bus operators in determining whether to purchase electric buses.

Although the results of the model suggest an energy-saving advantage enjoyed by electric buses, in view of the fact that the environment for deploying electric buses is far from mature in Taiwan, bus operators have yet to embrace this new technology.

Through the results of the investigation, this study aims to assist the government in formulating subsidy programs. We also hope that pilot demonstration projects will be carried out to encourage leaders in Taiwan's transportation industry to begin the deployment of electric bus fleets, which we believe will lead to widespread adoption of electric buses in this country.

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Using On-Road Data to Correct Underestimations of Fuel Consumptions of Motorcycles from Laboratory Tests

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ABSTRACT

There is not much literature on fuel economy performance of motorcycle in the real world. The mainstream official statistics report fuel efficiencies for each motorcycle model out of laboratory approval tests. However, our tests reveal that fuel efficiencies of urban arterials are 55~60% to the lab-test harmonic means, regardless age deterioration. Alternatively, with the on-board measurement system OBS-584 developed by ARTC suitable for motorcycles, it enables on-road data available for motorcycles. We found that average fuel consumptions by speeds show positive correlations to speeds, no matter of laboratory approval tests, urban or rural on-road tests. Moreover, similar correlation patterns of younger and older motorcycles suggest that the statistic regressions are probably a shared form among models or across ages. Further investigations into data directly collected from motorcycles in the real world might shed light on new approaches to better quantifying fuel consumptions of motorcycles driving on the road.

INTRODUCTION AND LITERATURE REVIEWS

In Asian urban areas, motorcycle is the most common private vehicle which is growing rapidly with economic prosperity. Motorcycle is often considered as an efficient mode with less fuel consumption; therefore the characteristics of fuel consumption of motorcycle in real world have not been seriously studied yet. There is not much literature on factors affecting fuel economy performance of motorcycle in real world. It leaves a major piece of fuel saving puzzles unknown in rapidly emerging Asian metropolitans.

The mainstream official statistics report fuel efficiency of each motorcycle model out of laboratory standard testing procedure. For example, Bureau of Energy (BoE), Ministry of Economic Affairs, publishes *Guidebook of Fuel Efficiency of*

Vehicles on the Market every year as the most detailed information regarding fuel efficiency among vehicles. The laboratory approval tests required by BoE follow similar procedures as the U.S. Environmental Protection Agency (U.S. EPA) generates yearly information for its *Buyer's Guide to Fuel Efficient Cars and Trucks*.

It is well known that the various driving patterns, ie. Driving Cycles in the required procedures of DoE or U.S. EPA, result in different fuel consumptions. Therefore, voluminous studies focus on forming “typical driving cycles” with local driving speed profiles in order to have laboratory tests closer to local realities. Chen, et al., (2003) and Tsia, et al., (2005) tried to sample driving cycles of motorcycle driving in actual urban and rural environment and develop representative driving cycles for laboratory tests. Zamboni, Carraro and Capobianco, (2011) investigated instantaneous speeds of motorcycles in various road classes to identify speed profiles most representing two-wheelers on the chassis dynamometer to improve the standard driving cycles. While it could be up to 20% of measurement deviations between the dynamometer of laboratory and real world driving conditions for a specific driving cycle (Chung-hua Institution for Economic Research, 2009). It is clear that in the laboratories it is not possible to replicate conditions of traffic flow and road configurations which significantly affects fuel consumptions. Even local driving cycles could not promise fuel consumptions out of laboratory tests are able to represent that of the real world.

Alternatively, owing to improvements of on-board measurement device, to have data directly from vehicles running on the road had been available since the last decades. It opens up a window toward estimations of fuel consumptions with data from the real world. Yet it is limited to heavy- and light- duty vehicles (Afotey, et al., 2013; Liu, et al., 2010; Tong, et al., 2000; Vojtisek-Lom and Allsop, 2001). Available commercialized systems mostly are designed for testing on light/heavy duty vehicles, namely as (1) Horiba OBS-2000 Series of Horiba Ltd., Japan; (2) AVL M.O.V.E On-Board Testing/ Gravimetric Filter Module of AVL, Austria; (3) AXionRS+NH3 of GLOBAL MRV, Inc., USA; and (4) SEMTECH-DS/ECOSTAR of Sensors Inc., USA. (Wen, et al., 2013). The on board measurement devices with credibility and reliability are not available for motorcycles regarding the size limitations and low weight capability of motorcycles.

This study takes advantages of the newly developed on-board measurement device specially designed for motorcycles to collect second-by-second data from motorcycles travelling on roads. With data from the real world, we analyze and test statistics significance then develop a set of parameters to correct underestimates from the results of laboratory.

EXPERIMENT DESIGN

1. On-board Measurement System: OBS-584

The on-board measurement system is crucial to this study. It has to be much more compact and lighter than available commercialized systems to be mounted onto motorcycles and consistently operating during road driving conditions. To have the desired on-board system for motorcycle on-road test, the research and development

team of the Automobile Research and Testing Center (ARTC) have been devoted in 3 years to realize the compact model we use onto the test motorcycles.

The system we use for motorcycle tests is named as OBS-584 which is the third generation prototype (see Figure 1). It is capable to record the synchronized second-by-second data of instantaneous speeds, emissions and engine working conditions from the motorcycle, locations, humidity and temperature of the environment. The major component of the OBS-584 is the analyzer of MEXA 584L of Horiba Ltd., Japan. Together with the gas flow meter, it is capable to provide instantaneous emissions data with reliability (IOT 2012). Moreover, OBS-584 is equipped with an engine sensor, humidity and temperature meter, the moving speed sensor onto the front wheel, and the global positioning system (GPS).



Figure 1. OBS-584 Installed onto One of Test Motorcycles

2. Test Routes

According to statistics released by the Ministry of Transportation and Communication (MOTC) in 2011, Taiwan has become the home of the highest motorcycle density in the world, with 419.3 motorcycles per square kilometer. The number of in-use motorcycles is more than 15 million in year 2012, resulting in more than 2 motorcycles per household in average. Except for Taipei, it is heavily depends on motorcycles as being with the most convenient and cost-effective one among modes of private transportation to serve diversified urban travel demand. Take Taichung metropolitan as an example, where is central to Taiwan with population over than 1 million in downtown and 3.5 million in metropolitan area, it is estimated that 50% or more trips is made by motorcycles for all trip purposes (MOTC 2010).

Given this backdrop, two typical roads in Taichung metropolitan area are selected to represent urban and rural arterials. As shown on Figure 2, the representative urban arterial is located in the very central downtown of Taichung city. On the urban arterial, street blocks are quite short and traffic flow is highly mixed with scooters, cars and buses. While a major road in the west outer area of the metropolitan is taken as the representative rural arterial. It is quite obvious from the map that blocks of the rural arterial are much longer than the urban arterial. The directions of traffic movement on the rural arterial are divided by raised crossing islands; and mixture of motorcycles from light/heavy duty vehicles is managed by pavement markings.

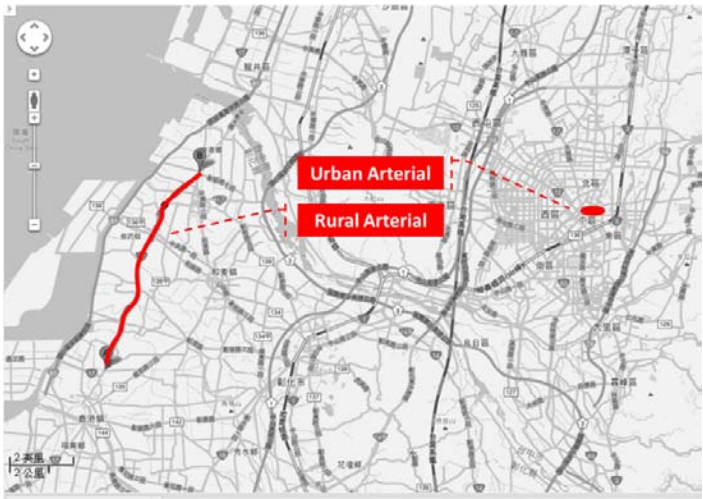


Figure 2. The Test Routes in Taichung Metropolitan, Taiwan

3. Test Motorcycles

We choose one 2.7-year KYMCO 125 c.c. and one 8.9-year SYM 125 c.c. to be our test models. The configurations of test motorcycles are listed on Table 1. The two models are the most dominant in the past 11 years as strong as 30~45% of the yearly new motorcycles fleet in Taiwan (see Figure 3). It accounts for 1.3 and 1.1 million for KYMCO and SYM respectively, out of 7.6 million of total sales during year 2002 to 2012. According to the most updated MOTC survey (MOTC 2012), the average age of in-use motorcycle fleet of Taiwan is 10.5 years old. Therefore we believe the KYMCO 125 c.c. and the SYM 125 c.c. are well representatives of in-use motorcycles in Taiwan. We also expect the trends of fuel consumptions we found here are highly possible to be transferred to nearby Asian areas; for the reason that the powertrain technologies on both test motorcycles are similar to those popular types sold in Asian countries.

Table 1. Characteristics of Test Motorcycles.

	KYMCO	SYM
Engine Type	Single Cylinder, 4 Stroke	Single Cylinder, 4 Stroke
Dry Weight Amount (kg)	110	104
Displacement (cc)	124.6	124.6
Max. Horsepower (ps/rpm)	9.6/7,500	10.5/8,500
Max. Torque (kg-m/rpm)	0.98/6,500	1.00/6,500
Transmission System	CVT	CVT
fuel supply system	Fuel Injection	Carburetor
Bore*Stroke (mm)	ψ52.4×57.8	ψ52.4×57.8
Fuel Capacity (L)	6.0	6.0
Homologation(R.O.C.)	Phase 5	Phase 4
Age (year)	2.7	8.9
Travelled Distance (km)	9,815	31,352

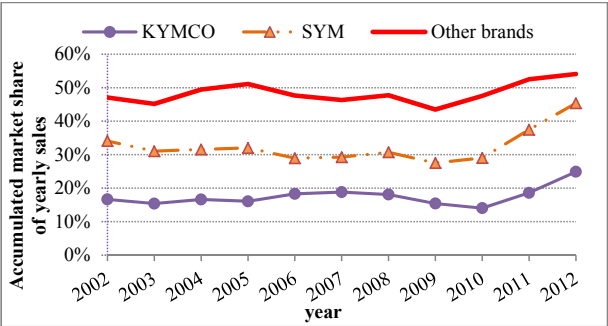


Figure 3. The Market Share of 125 c.c. Between Year 2002 to 2012, Taiwan

4. Data Collections

We collected data from test motorcycles with OBS-584 identically to each road type. Each motorcycle is paired with one specific team member for on-road tests to avoid driver’s deviation. Test drivers are subject to follow the traffic flow in morning and evening rush hours in non-raining weekdays. On-road tests were conducted during May to July 2012 and 2013. The data collected counts for more than 1.2 million seconds for each road types for both models, with driving distance among 70~160 km.

To compare with results out of laboratories, fuel efficiency approval tests in laboratory of two test motorcycles are executed by ARTC which is an authorized laboratory for DoE fuel economy approval tests. Tests in laboratory strictly follow the required procedures and specifications according to the model year of the test motorcycle. It is repeated three times to get sufficient data for statistics analysis.

RESULTS AND DISCUSSIONS

The published fuel efficiencies and counterparts of our tests are listed on Table 2. Fuel efficiencies of lab approval tests of two test motorcycles, executing before road tests, are 80~96% to the published numbers shown on the BoE yearly books. The older SYM performs 5~6 % lower than the younger KYMCO. It confirms that age deterioration leads to lower fuel efficiencies as well that the older the more noteworthy of deterioration. Furthermore, for each test motorcycle, deterioration rations of urban cycles are significantly lower than of cruise at 50 kph. It might imply that urban cycles with accelerations and decelerations retain more deterioration effects than that of cruise at 50 kph.

Results from the urban arterial road tests of KYMCO and SYM show only 53% and 46% of the published harmonic means respectively. It indicates that the fuel efficiency of daily commuting on urban arterials will be roughly as half as people learn from the BoE yearly books; and the older SYM performs 7% lower than the younger KYMCO. Regarding accelerations and decelerations arise more often on urban arterials in the real world than of urban cycles of the approval tests, it echoes the previously mentioned statement that driving cycles with more accelerations and

decelerations might retain more deterioration effects.

Table 2. Fuel Efficiencies.

		KYMCO			SYM			
		Fuel Efficiency (km/l)	Percent to test of model year	Percent to published harmonic mean	Percent to lab-test harmonic mean	Fuel Efficiency (km/l)	Percent to test of model year	Percent to published harmonic mean
Published Approval test (model year)	Urban cycle	42.40				39.02		
	Cruise at 50 kph	57.90				53.04		
	Harmonic Mean	47.40				43.66		
Lab approval test (test year)	Urban cycle	36.34	86%			31.27	80%	
	Cruise at 50 kph	55.75	96%			48.44	91%	
	Harmonic Mean	42.23	89%			36.40	83%	
On-Road test	Urban arterial	25.13		53%	60%	19.95		46%
	Rural arterial	42.28		89%	100%	38.54		88%

Note: numbers shown on “approval test (model year)” are from BoE(model year).

Based on results of our tests, the on-road fuel efficiencies of urban arterial is 60% and 55% to the lab-test harmonic means for KYMCO and SYM respectively. It leads to potential parameters to correct underestimations of fuel consumption from approval tests in laboratory; though more tests are necessary to confirm to confirm correction parameters with credibility and reliability.

Following Wen, et al., (2013), we calculate average fuel consumptions for each speed of all tests then plug the results onto Figure 4. It shows positive correlations of averaged fuel consumptions to speeds, no matter in laboratory approval test, urban or rural on-road test. Those similar correlation patterns of average fuel consumptions to speeds among tests or across models suggest that fitted regressions with statistically significance are highly possible, as demonstrated on Wen, et al., (2013). Therefore, it is promising to investigate data collected from the real world to approach quantified estimations of fuel consumptions of motorcycle driving on the real world.

CONCLUSIONS

With the on-board measurement system OBS-584 developed by ARTC suitable for motorcycles, it enables on-road data available for motorcycles. Following Wen, et al., (2013), we calculated average fuel consumptions by speeds and found that average fuel consumptions by speeds show strong positive correlations to speeds, no matter of laboratory approval tests, urban or rural on-road tests. Moreover, similar correlation patterns of younger KYMCO and older SYM suggest that the statistic regressions proposed by Wen, et al., (2013) are probably a shared form among models or across ages. Further investigations into data directly collected from motorcycle in the real world might shed light on new approaches to better quantifying fuel consumptions of motorcycles driving on the road.

While back to the mainstream official statistics of laboratory approval tests, the on-road fuel efficiencies of urban arterials are 60% and 55% to the lab-test harmonic means for KYMCO and SYM respectively, regardless age deterioration. It

raises potential parameters to correct underestimations of fuel consumptions from approval tests in laboratory; though more tests are necessary to confirm correction parameters with credibility and reliability.

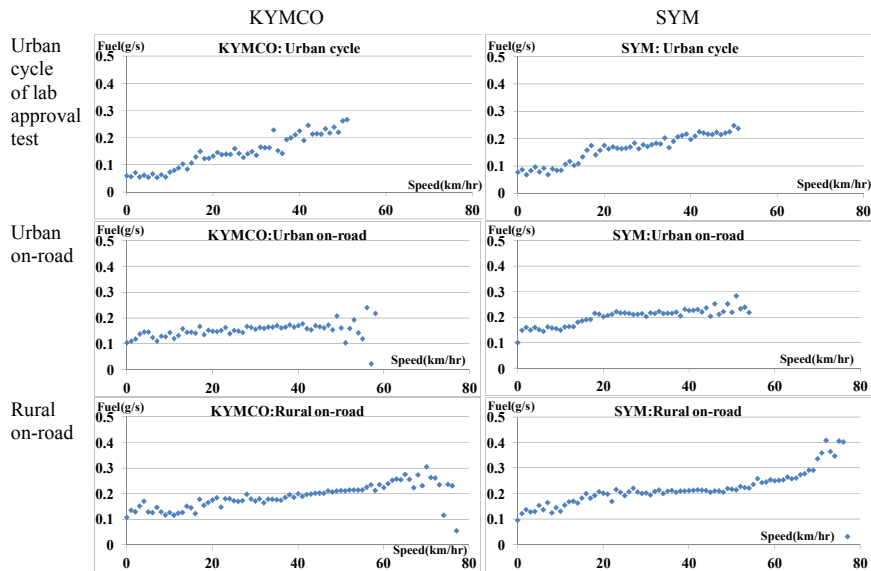


Figure 4. Fuel Consumptions by Speeds.

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Analysis of Impacts on Environment from Electric Bicycles: Case Study of Xi'an, China

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ABSTRACT

The increasing popularity of the use of electric bicycles in China recent years has transformed the urban traffic travel structure, changing the impacts on natural environment from traffic system. By analyzing the energy consumptions and emissions from various traffic modes during life cycle, this paper discussed the impacts (energy consumption, carbon emissions, SO₂, PM and pb consumption) on environment from the transformation of urban travel behaviors caused by the development of electric bicycles. Taking advantage of the electric bicycle travel survey data in Xi'an, 2012, results have been found that the use of electric bicycles has increased the emissions of SO₂, PM and the consumption of pb, reduced the energy consumption and CO₂ emissions, however.

KEY WORDS: electric bicycles, environment, life cycle theory

1. INTRODUCTION

According to statistics, in 1998-2008, electric bicycles in Chinese towns maintained a high growth rate with 80%. By the end of December, 2008, Per hundred urban household have 23.1 electric bicycles on average in China (Liu 2010; Xu 2011), which became an important part of travel pattern in Chinese cities and towns. Since China's urban development is facing the great pressure from traffic congestion, energy shortage, environment pollution and so on, to analysis the development of electric bicycle by the view of transportation energy and environmental impact has important significance for formulating developmental policy of electric bicycle in cities and guiding the development of the electric bicycle in other cities in the world.

The researches existing currently mainly center around the characteristics of travel behavior by electric bicycle, the effect from electric bicycle on urban traffic configuration, the electric bicycle's energy consumption and environmental pollution, etc. As a new way of transport, electric bicycle attracts some travelers

from their original way to travel. However, research on the effect of this behavior on the natural environment is limited. Electric bicycle has been an important part of travel pattern. The travel rate by electric bicycle reaches 10% in Xi'an, China, which means about 1 million people travel by electric bicycle every day. So it's obvious that the impact energy consumption and pollutant emission which are caused by electric bicycle on natural environment. Understanding the influence relations will contribute to realizing the merits of electric bicycle, the impact on urban traffic and environment more clearly.

Proceeding from this point, using this survey data about the electric bicycle in Xi'an, China, this paper analyzed the impact the travel behavior change caused by the development of urban electric bicycle on the natural environment (energy consumption, carbon, sulfur dioxide, total suspended particulate matter, lead consumption, etc.) based on the theory of whole life cycle.

2. LITERATURE REVIEW

2.1 Travel characteristics of electric bicycle

Cherry (2007) analysis the factors affecting travelers choose electric bicycles and users' choice intention of a variety of way to travel; Liu (2008) and Liu (2011) analysis the change rule of the price of electric vehicles, battery recycling situation and people's age, income, occupation with travelers' data in Shanghai; Dong (2008) analysis traffic characteristics of electric bicycle such as travel purpose, distance and the quantitative relations between non-motor vehicle lane capacity and electric bicycle rate with the electric bicycle data of Shanghai.

2.2 Impacts on urban traffic configuration from electric bicycle.

Through the comparative analysis of the characteristics and cost efficiency of various modes of transport, Yang (2005) and Zhou (2005) think electric bicycles will be play a dominant part in the urban traffic structure in China; Weinert (2006) surveys bike and e-bike users in Shijiazhuang and finds that E-bikes are enabling people to commute longer distances. This has important implications on energy use, accessibility and urban expansion of cities. People under-served by public transportation are shifting to e-bike. By researching the travel data of Kunming and Shanghai, Cherry (2007) find most of users will shift to public transit if they can't use electric bicycles. Electric bicycle is not only a traditional motorized transportation between bicycles and cars, but a motorized transportation with cheaper cost and better service than the public transportation. By researching the travel data of Chengdu and Shanghai, Liu (2011) statistical analysis of the ban if the electric bicycle, electric bike users transfer rule.

2.3 The Electric bicycle's energy consumption and environmental pollution

By the study of electric bicycle's energy consumption and environmental pollution, Liu (2002) and other domestic scholars think the electric vehicles is a kind of green and ecological mode of transportation; with the travel data of electric bicycles in Shanghai and Kunming, Kanafani (2007) and Cherry (2009) analysis the electric bicycle's pollution based the theory of whole life cycle and find that the electric bicycles' emissions will be far less than motorcycles, cars per mileage, which is similar to bus and higher than the bicycle.

The extent research studied the electric bicycle's environmental pollution and the effect from electric bicycle on urban traffic configuration. However, since different kinds of transportation means have different energy consumption, research on the effect of this behavior on the natural environment is limited. This paper, taking xi 'an as an example, analyzed the relationship of shifting between electric bicycle and other existing traffic modes. From the view of energy consumption and pollutant emission in the whole life of travel means, this paper quantify the impact the travel behavior change caused by the development of urban electric bicycle on the natural environment.

From 2008 to 2011, the rate of travel by electric bicycle in Xi'an increased from 4.55% to 4.55%. By the end of 2011, Xi'an, as a typical fast-growing private motorized city, had about 650000 electric bicycles. The electric bicycle's development in Xi'an has being in a free mode without policy intervention, which provide good environment to understand the shift intention of electric bicycle users.

3. LIFE CYCLE THEORY

All life cycle refers to the whole process as "from cradle to grave" of a product, including all phases as introduction, growths, maturity, saturation and decline. As for traffic modes, four stages exist likewise: production stage, circulation stage, usage stage and recycling stage. It directly consumes energy and produces pollutants during usage stage, causing environmental impacts on cities, however, it's far from enough to just take usage stage into consideration when comparing the energy consumption and emissions among all traffic modes. Taking electric bicycles as example, the energy consumption during usage stage account for 50 percent in life cycle, 40 percent for CO₂ emissions, and almost no solid waste or wastewater produced during usage. However, it consumed a lot of energy and produce pollutants during other stages, so it necessary to compare the energy demands and pollution emissions of various traffic modes from all life cycle perspective. Kanafani (2007) and Cervero (2009) have studied the energy consumption and emissions during life cycle for all traffic modes in Shanghai and Kunming, China. The results are listed in Table 1.

Table 1. Energy and Pollution Emissions of Various Traffic Modes during Life Cycle

	energy use (kwh/100 pax-km)	co ₂ (g/pax-km)	so ₂ (g/pax-km)	Pm (g/pax-km)	Co (g/pax-km)	Hc (g/pax-km)	no _x (g/pax-km)	Pb (mg/pax-km)
car	47-140	102-306	0.23-0.69	0.09-0.28	3.4-10.1	0.57-1.67	0.44-1.32	0.018-50.03
bus	8.7-26.2	24.2-96.8	0.01-0.04	0.04-0.14	0.08-0.32	0.008-0.03	0.14-0.54	0.001-0.004
motor- cycle	21-42	64-128	0.04-0.08	0.2-0.4	6.3-12.5	1.13-2.25	0.08-0.15	0.016-0.032
bicycle	4.88	4.7	0.01	0.06	unclear	Unclear	Unclear	0
Electric bicycle	3.8-7.6	15.6-31.2	0.07-0.14	0.07-0.14	0.007-0.014	0.027-0.053	0.01-0.02	0.145-0.290
Electric motor- cycle	4.9-9.9	20.2-40.5	0.09-0.17	0.1-0.19	0.009-0.017	0.032-0.064	0.014-0.027	0.210-0.420

4. CASE STUDY

4.1 Data Collection

As one of the four historic ancient civilized cities, Xi'an located in the heart area in northwest China, with 369 km2 built area, and 8.43 million resident populations. Up to 2011, Xi'an has about 650,000 electric bicycles, sharing more than 10 percent in urban transportation, has become very important to people's daily trip transportation. We take the data from "Electric Bicycle Travel Survey in Xi'an" launched in April, 2012. Survey objects contain electric bicycle users and nonusers; the survey measure was stopping electric bicycle users on the road and filling the questionnaire face to face; survey content covered the electric bicycle users' characteristics(travel purpose, time, frequency), bicycle characteristics(purchase price, age of the bicycle, maximum speed), travel information(traffic mode taken before electric bicycle, the alternative traffic mode provided giving up electric bicycles); survey area targeted in the built area within the third ring, totally, 600 questionnaires were distributed each to electric bicycle users and non-users, and actual valid questionnaires returned were 576 and 599.

4.2 Transfer Law Analysis of Electric Bicycles

The development of electric bicycles has increased the traffic modal alternatives for daily travel in cities, transforming the original transportation mode shares.

(1) Status sources of electric bicycle users

The sources of electric users mainly from two parts: one is transfer from other traffic modes; the other is the induced travel by the advantages of electric bicycles. According to the analysis of the survey data, among all the users, about 8 percent were induced travel users (see Table 3), all the others transferred from other modes. More than 85 percent come from bus, bicycle and walk; 2.6 percent from car users, 2.9 percent from taxi takers; and 4.3 percent from motorcycle users due to the motorcycle forbidden rule within the second ring in Xi'an (see Table 2).

Table 2. Transfer Rates from Other Modes

Traffic modes	Walk	Bicycle	Motorcycle	Bus	Car	Commute shuttle	Taxi	Metro	Others
Transfer rate	16.3%	31.4%	4.3%	38.3%	2.6%	0.4%	2.9%	1.0%	2.9%

Table 3. Induced Travel or Not?

Induced or not	No	Yes
Proportion	92%	8%

(2) Electric bicycle users future transfer desire analysis

The travelers who take electric bicycle as the first choice now may abandon it and transfer to another mode in future when income increase and economy develop. According to the survey, 27 percent users may transfer to other modes (see Table4), out of which, the main target is car, and bus next.

On the other hand, due to the advantages of electric bicycles and high gasoline price, non-users of electric bicycles may abandon the status choice and transfer to electric bicycles. According to the survey in Xi'an, there were 38 percent of non-users may transfer to electric use (see Table6). The car users won't transfer to electric bicycles, the transfer sources still come from bus passengers (56.8%) and slow traffic (31.4%) (see Table7).

Table 4. Transfer Intention of Electric Bicycle Users

Will transfer or not	Will	Won't
Proportion	27%	73%

Table 5. The Intentions of Transferring To Other Modes from Electric Bicycle

Traffic modes	Walk	Bicycle	Bus	Car	Metro	Commute shuttle	Taxi	Unclear
Transfer rate	10.5%	5.2%	19.6%	47.1%	9.8%	2.6%	3.3%	2.0%

Table 6. Transfer Intention for Non-Users of Electric Bicycle

Transfer intention	Yes	No
Proportion	38%	62%

Table 7. The Status First Mode Choice of The Transfer Intenders Out Of Electric Bicycle Non-Users

Traffic modes	Walk	Bicycle	Motorcycle	Bus	Commute shuttle	Metro	Taxi
Proportion	17.8	13.6	2.5	56.8	1.7	5.9	1.7

4.3 Impact Analysis on the Natural Environment of Electric Bicycles

Assuming there were no electric bicycles, travelers choose the original traffic mode according to the transfer rate obtained from the survey. We could compare the pollutant emissions under the two circumstances of hypothesized circumstance without electric bicycles and the real situation with electric bicycles.

(1)Travel transfer amount analysis

Assuming the electric bicycle daily turnover is M in Xi'an, 2012, among which 8 percent was induced demand, 92 percent were transferred from other modes. Thus the potential travel quantities of each mode were as followed (see Table 8):

Table 8. Travel Transfer Amount

Traffic modes	Transfer amount
Walk	$16.3\% \times 92\% \times M$
bicycle	$31.4\% \times 92\% \times M$
Motorcycle	$4.3\% \times 92\% \times M$
Bus	$38.3\% \times 92\% \times M$
Car	$2.6\% \times 92\% \times M$
Commute shuttle	$0.4\% \times 92\% \times M$
Taxi	$2.9\% \times 92\% \times M$
Metro	$1.0\% \times 92\% \times M$
Others	$2.9\% \times 92\% \times M$
Induced	$8\% \times M$

(2)The extent of impact on environment

Taking use of the emission difference between the cases of without and with electric bicycles use, we could figure out the change caused by electric bicycles.

The emission factors of CO, HC and NOx from bicycles are unclear, so here we ignore these three pollutants in analysis. If value ΔT_i was negative, suggesting the development of electric bicycles help to reduce harmful emissions, if ΔT_i was

positive, means electric bicycles increased the pollution (see Table 9).

$$\Delta T_i = \delta_{ei} * M - \sum_n (L_n * \delta_{ni} * R * M)$$

Where, ΔT_i represents the emission difference of pollutant i between the two situations of hypothesized case without electric bicycles and the status case with electric bicycles. L_n is the rate of electric bicycle users transferring from traffic mode n; δ_{ni} is the emission factor of pollutant I from traffic mode n; δ_{ei} is the emission factor of pollutant I form electric bicycle; R is the non-induced rate for electric bicycle; M is passenger turnover of electric bicycles

Table 9. The Change Extents of Various Pollutants and Energy Use

	energy use	co ₂	so ₂	pm	pb
ΔT_i	-5.797*M	-9.82*M	0.044*M	0.031*M	0.197*M

From the perspective of life cycle analysis, the popular usage of electric bicycles reduced the energy consumption and CO₂ emissions, and increased the emissions of SO₂, PM and pb consumption. Referring to the 2011survey data, the daily travel turnover was 5.88 million (person.km/d), so the quantity changes of various pollutants caused by electric bicycles in Xi'an were obtained. Further, we collected the total energy consumption and various pollutant emissions in China, analyzed the changing extent of energy consumption and emissions in Xi'an to the national total (see Table10).

Table 10. Change Extents of Various Pollutants and Energy Use Of Xi'an

	Energy use(kwh)	co ₂ (t)	so ₂ (t)	pm (t)	pb (t)
ΔT_i	-1.2*10 ⁶	-2.1*10 ⁴	93	66	420
National total	41239*10 ⁸	75*10 ⁸	2.2*10 ⁷	8.3*10 ⁶	350*10 ⁴
Extent	-0.3*10-6	-2.8*10-6	4.23*10-6	7.95*10-6	120*10-6

5. DISCUSSION

Take Xi'an as an example, from the perspective of life cycle, electric bicycles mainly replaced the modes of bicycle or walk, the advantages of low energy consumption and few emissions were diluted, increased the emissions of SO₂, PM and consumption of pb, the energy consumption and CO₂ emissions were reduced, however.

Through the comparison of increase or decrease amount of pollutants to national total, it's found that the change extent of SO₂, PM, pb were higher than energy consumption and CO₂ emissions.

Assuming electric bicycle mode keeps develop freely, the transfer sources

may still be the bus passengers, bicycle riders and walkers, car drivers almost show no transfer desire, In future, electric bicycle users would transfer to car. This changing trend of travel structure may not be helpful to environment.

6. CONCLUSION

This paper analyzed the impacts on natural environment from travel mode transfer caused by the development of electric bicycles from the perspective of life cycle. Taking advantage of the survey data in Xi'an, two points were found:

1) Electric bicycles mainly replaced the modes of walk (16.3%), bicycle(31.4%), bus(38.3%) and car users(2.6%); if electric bicycle users try to abandon it, the majority will transfer to car use(47%), making the congestion even worse and increasing energy consumption.

2) From the perspective of life cycle, it's found that electric bicycle replaced large quantity of slow traffic travel modes, reducing energy consumption (-0.5×10^{-6}), CO_2 emissions (-2.8×10^{-6}); increasing the emissions of SO_2 (3.72×10^{-6}), PM (4.3×10^{-6}) and use of pb (120×10^{-6}), and the change extent of the latter three kinds were higher than the former two.

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Implementation of Urban Green Transportation: Public Bikes in Taipei and Kaohsiung

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ABSTRACT

This study analyzes urban development and green transportation design for Taipei and Kaohsiung, Taiwan. Traffic usage characteristics are analyzed, the public bike development experience of the two cities is compared, and, finally, a reference for other cities to develop green transportation is proposed.

INTRODUCTION

In the past 20 years, developing countries or transition economies have seen dramatic increases in urbanization and motorization. Major metropolitan areas have also faced big challenges in traffic congestion, noise and air pollution, energy consumption, and traffic safety, mainly caused by car- or highway-oriented development approaches. Economic loss and public health threats due to traffic congestion and pollution have significantly affected living quality in major cities.

These critical challenges have been recognized in central and local governments, and various policies related to sustainable urban development and green transportation have been proposed.

With the increasing in transportation supply, transportation demand management (TDM) has become more important. Transportation sharing has been a new traffic management approach in recent years. Under the existing transport

supply, shared transportation modes make transportation more effective and avoid waste and vacancy. Transportation sharing has many types, including public transportation (bus, metro, and taxi) and car- and bike-sharing. For cities, public transportation has aided in the implementation of sustainable development in the world. Because the concept of car-sharing must cause changes to habits of using private transportation modes, it is still in the beginning stages. In pursuit of the goal of a livable city, slower transportation modes are receiving increased attention, and the bicycle traffic is increasing in comparison to other transportation sharing types. The speed of implementation of biking is faster, and it also meets the need of the “last mile” in trip chain, so it is widely welcomed in cities.

In this study, we first discuss the relationship between urban development and green traffic and then analyze the green transportation development process in the past and future development plans in Taipei and Kaohsiung, especially focusing on the analysis of their public bike development experiences and comparing the advantages and disadvantages of the development process. Finally, conclusions for other cities can be used as a reference for developing public bicycles.

CITY BACKGROUND

Taipei and Kaohsiung are the two largest metropolises in Taiwan and are in the southern and northern economic centers, and led to the development of the outbound and inbound regions. Therefore, in urban transport planning, it is necessary to not only consider the urban center metro rapid transportation modes, but the need to plan the “last mile” of user trips to ensure that the overall trip is smooth and convenient.

The total area of Taipei currently was 272 square kilometers at the end of October 2013, and the population was about 2,720,000. There were about 7.2 million cars and 1.05 million motorcycles; the MRT route length was 121 km with 4,000 buses. Because of the limited area of urban roads and parking, before the MRT opened, traffic congestion was very serious, and Taipei became the only one city to reach more than a 40% public transportation utilization rate in Taiwan (which, overall, is about 14% in Taiwan), and green traffic usage was up to nearly 60%.

Relative to the high usage of public transportation of Taipei, Kaohsiung is located in southern Taiwan and has a total area of approximately 2,946 square kilometers. The CBD (old Kaohsiung City) area is approximately 154 square kilometers with a population of about 1.5 million. Due to the population density being much lower than Taipei, it is very convenient to park, so public transport usage

is very low, at only about 10%. This is mainly due to the length of the average trip times of about 21 minutes in Kaohsiung and less than the 40 minutes in the Taipei metropolitan area. Residents use private modes to complete the trip chain. Under the sustainable policies of the government in recent years, the urban development form of the industrial city in the past became a service and tourism-oriented, and thus led to the rise of green transportation. Green transportation usage gradually is rising due to the reform of the Metro and the bus system, and another key factor is the implementation of public bicycles.

SYNTHESIS OF PUBLIC BIKE FEATURES

Design elements. Related to bike lane facilities, comparing bicycle lanes in the Taipei and Kaohsiung systems, currently in Taipei has completed 111 km of riverside building bike lanes, which includes 34.8 km of urban bicycle lanes and 91.2 kilometers of pedestrian and bicycle shared road. In Kaohsiung, the current bike lanes are 500 km and are expect to be 600 km by 2014.

Related to bike facilities, demonstration operations began in March 2009 in Taipei, which initially set up 11 stations and provided 500 bikes in Xinyi. Because the rental stations were not widespread, the utilization rate was not high; early usage was only 180 times per day. After the whole system was officially opened in November 2012, Taipei public bike (YouBike) had 130,000 members, and the cumulative number of uses was over one million. As of November 2013, there were 124 rental stations in 12 districts in Taipei, and 162 sites were expected to be completed before the end of 2013. Public bike rental stations were located in transfer stations or MRT stations, the surrounding residential and commercial area. Integration of public bikes and MRT, buses, and other public transportation systems improved the overall public transport network.

The public bike rental system (City Bike) in Kaohsiung also began operating in March 2009 and is a commercial public bike rental system managed by the Kaohsiung Rapid Transit Corporation to take up the last mile of transit services. City Bike is the first metropolitan network public bicycle rental system in Taiwan, initially laid with more than 50 sites. Because of the high bike lane density, it meet the needs of the public on holidays; the usage is higher than in Taipei. The system is in competition with Kyoto, Jeju, Beijing, and Singapore for the best bike-riding Asian cities, as cited by CNN. Currently, the system provides 7,000 bikes (with actual operation of approximately 1,000) and 123 rental stations in Kaohsiung.

In operational terms, Taipei YouBike is 24-hour unattended self-service, and each site has an automatic service machine (kiosk). Users can use the service machine to apply for membership, pay for, and rent bicycles. The leasing model is divided into members and non-members, and a membership application can be completed on the YouBike website, at an automated service machine, or at service centers. Each lease parking site is equipped with smart-card sensor devices, and registered members can directly use their smart card to take a bicycle. Non-members must use the automatic service machine (smart card, credit card, or mobile phone number payment), then take the bike within 90 seconds. Rental costs is NT \$10 for 30 minutes. YouBike's progressive rates remain at NT \$10/30minutes for up to 4 hours 30 minutes; the rate for 4–8 hours increases to NT \$20/30 minutes, and for more than 8 hours, the rate is NT\$ 40/30 minutes. Since receiving government grants, membership is free in the first 30 minutes.

The public bike rental stations in Kaohsiung are also operational for 24 hours a day and accept smart cards and VISA, Master Card, and JCB credit cards. The credit card kiosk is different from YouBike. It is the first system in which users can rent a bicycle at one station and return it to another. To encourage use, rental rates for the first hour is free, then the rate is NT \$10/30minutes after that (or NT \$20/30minutes if using a credit card).

Operational performance. The Taipei YouBike system was officially launched in March 2009, and by the end of November 2013, there were more than 8.3 million users, primarily due to the impact of rental sites and a variety of marketing activities. In 2013, the number of rental sites increased from 48 to 108. The total number of bicycles increased from 1,700 to 3,600, and the number of users increased from 0.28 million to 1.25 million by September 2013, a growth rate of 400%. Since many popular rental sites often lack bicycles, to increase use, the bicycle rental rates were adjusted slightly in October 2013.

In Kaohsiung, by the end of October 2013, there were more than 3.1 million users, and an increase of 0.13 million passengers in December 2012 to 0.25 million users, a nearly 100% increase. Public bike use obviously has gradually been accepted by the people. In 2011, the Kaohsiung Rapid Transit Corporation took over the business and integrated the use of smart cards with free transfer to the MRT, increasing the number of MRT trips indirectly. A comparison of the public systems in Taipei and Kaohsiung is shown in Table 1.

Table 1. Comparison of Taipei and Kaohsiung Public Bicycle Systems

Characteristic	Taipei	Kaohsiung
Implementation scope	Metro line and transportation center	Metro line and park
Rental stations	124	123
Bicycles	5,350	7,000
Stations and line serve areas	80%	70%
Distance between 2 stations	Close	Far
Rental convenience	Easy	Easy

KEY FACTORS FOR SUCCESS

Development of a public bicycle system can help a society pursue goals of sustainable development and approach the achievement of green energy and carbon reduction. From the case study of Taipei and Kaohsiung, from a policy perspective, public bicycles must be effectively integrated with public transportation and can be achieved through a variety of marketing and promotional activities to attract users. From a technical perspective, it must be combined with smart cards to facilitate use and should combine the ecological environment and recreation to achieve the transformation into the cultural environment.

According to our analysis, some key factors in planning a successful public bicycle system are as follows:

- Determination and promotion of the government: good bike system planning and control signals, integration with public transportation system.
- Long-term business of management team: easy rental mode, reasonable costs and all-day service, extensive and in-depth marketing.
- Public support: good transport facilities supported by the public to enhance utilization.
- Improvement with expert guidance: good traffic engineering, management, and operational methods, experience of international experts.

CONCLUSIONS AND RECOMMENDATIONS

Public bicycle sharing is an important element in transportation sharing and allows users to complete an important link in the trip chain. This study analyzed the Taipei and Kaohsiung public bike systems and presented the key success factors related to

rental stations sites, convenience, and marketing. After a public bicycle system is developed, there still may be some issues, such as system maintenance, vehicle demand and scheduling, etc., that will affect system performance if expanding the scale of the operation.

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Economic Analysis of Highway Design Exception Projects

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ABSTRACT

For various reasons, it may be necessary to design a section of highway with substandard values for some elements. Such design exceptions require appropriate assessment and justification of the potential impacts to highway safety and operations. It is necessary to conduct economic analyses for proposed design exception projects to compare with the standard designed projects. An economic analysis method was developed to conduct benefit-cost evaluations for design exception highway projects. The method considers life-cycle-costs as well as life-cycle-benefits resulted from a proposed built project in terms of monetary values. Agency costs are estimated by engineers or designers based on past experience, bid prices, design specifications, materials costs, and other information. Travel time and delay costs are usually valued as a percentage of average personal wages. Accident or crash costs are the monetary values for fatalities and injuries associated with crashes. The vehicle operating costs are attributed to the changes in highway speeds, traffic congestion, pavement surface, and other conditions that affect vehicle fuel consumption and wear and tear. With this analysis method, engineers will be able to conduct benefit-cost analysis efficiently and accurately for design exception projects.

INTRODUCTION

The American Association of State Highway and Transportation Officials (AASHTO) publication titled “A Policy on Geometric Design of Highways and Streets” (Green Book) (AASHTO, 2001) provides guidance to the designer by referencing a recommended range of values for critical dimensions for the design of highway

facilities. For various reasons, it may be necessary to design a section of highway with substandard values for some elements. Such design exceptions require appropriate assessment and justification of the potential impacts to highway safety and operations. This study was conducted to develop methods for assessing highway design exception projects for the Indiana Department of Transportation (INDOT).

One of the important reasons for design exceptions is to reduce cost. It is necessary to conduct economic analyses for proposed design exception projects to compare with the standard designed projects. However, when conducting economic analysis, not only the agency costs (design and engineering cost and construction cost) should be considered, but also the user costs and benefits should be considered. An economic analysis should consider all costs as well as benefits resulted from a proposed built project in terms of monetary values. One of the economic analysis methods is life cycle cost analysis (LCCA). It is a useful economic tool for selecting among alternatives where benefits of the possible project alternatives are essentially identical. For design exception projects, the benefits of alternative projects are usually not the same. Therefore, LCCA method is not suitable for evaluating design exception projects. The appropriate economic tool for design exception projects is benefit-cost analysis (BCA), which considers life-cycle benefits as well as life-cycle costs (FHWA, 2012).

SAFETY EVALUATIONS OF HIGHWAY FEATURES

A major concern for design exception projects is their potential effects on roadway safety due to the features that are designed below the standard requirements. Therefore, it is essential to be able to evaluate the safety impact of design exception projects. The newly published Highway Safety Manual (HSM) (AASHTO, 2010a) provides comprehensive and powerful tools for evaluating safety effects of various types of highway projects. Also available is the Interactive Highway Safety Design Model (IHSDM) (FHWA, 2010a), which is a suite of software analysis tools for evaluating safety and operational effects of highway projects with respect to geometric design characteristics. IHSDM was developed in coordination with HSM. Most of the HSM procedures and mathematic equations are incorporated into the IHSDM software package, so that the necessary and tedious computations are computerized.

HSM provides an array of tools for roadway safety analysis. The current edition of the HSM provides comprehensive methodologies on measuring, estimating and evaluating roadways in terms of number of crashes and crash severities. HSM can be used to identify highway sections with potential safety problems, the factors contributing to these safety issues, and the potential countermeasures to address these issues. It can also be used to conduct economic appraisals of proposed improvements and to evaluate safety benefits of proposed or implemented treatments. HSM provides effective tools for highway engineers and planners to make appropriate and rational decisions on various types of highway projects. The HSM tools are useful in all the stages of highway projects, including system planning, project planning and preliminary engineering, design and construction, and operations and maintenance.

An important and useful part of HSM is the crash prediction capability. The crash prediction methodology can be used with or without site-specific crash history

data. When crash data is available for a specific highway site, the HSM method incorporates the site specific crash history and produces crash predictions suitable for the given site. If the crash data is not available for a highway site, HSM will use the representative crash data for the type of highways based on the national database of crashes for the analysis. This application is usually for planned roadways that have not yet been constructed as well as for an existing roadway without site-specific crash history data. To analyze the safety effects of a given highway section, the HSM crash prediction model first applies general safety performance functions (SPFs) for a baseline condition and then adjusts the safety performance using a set of crash modification factors (CMFs). The crash modification factors are used to adjust the safety measures to a particular jurisdiction or geographical area.

HSM offers specific equations for different types of highways, for the purpose of introduction and demonstration, only presented in the following are the set of equations for undivided two-way highways. All the equations and the data presented in this section are from HSM (AASHTO, 2010a). The general form of the HSM crash prediction is based on the following equation:

$$N_{rs} = C_r \times N_{spf-rs} \times (CMF_{1r} \times \dots \times CMF_{nr}) \quad (1)$$

where:

N_{rs} = Predicted number of crashes per year;

N_{spf-rs} = Predicted number of crashes per year for nominal or baseline conditions;

C_r = Calibration factor for a particular jurisdiction or geographical area;

CMF_{nr} = Crash modification factors for roadway segments.

The equation for the predicted number of crashes for baseline condition is:

$$N_{spf-rs} = AADT \times L \times 365 \times 10^{-6} \times e^{-0.312} \quad (2)$$

where:

ADT = Average daily traffic (veh/day) on roadway segment;

L = Length of roadway segment (mile).

The baseline conditions for the crash predictions are specified as 12 ft of lane width, 6 ft of shoulder width, roadside hazard rating of 3, driveway density of 5, no horizontal curvature, no grade, no centerline rumble strip, no auxiliary lane, no lighting and no automated speed enforcement. The coefficients in the above equation are replaceable by users coefficients if they have those values calculated for their jurisdictions. Crash modification factors are provided in HSM for the actual conditions that are different from the specified baseline conditions. The crash modification factors include those for traffic volume and composition, lane width, crash type, shoulder width, shoulder type, horizontal curvature, superelevation, vertical grade, driveway density, centerline rumble strip, passing lane or climbing lane, two-way left-turn lane, roadside hazard rating, roadway lighting, and speed enforcement.

The software package, IHSDM, incorporates all the specific formulas and default modification factors so that the appropriate default modification factors are automatically selected in the computation process. In the case that the user has his/her own modification factors for given conditions, the user can input these factors to overwrite the default values. The software provides a step-by-step instruction, called Wizard, for users to follow. The Wizard leads a user from data input to evaluation output.

APPLICATIONS OF IHSDM

To demonstrate the process of safety evaluation with IHSDM, an example problem is presented in the following. A section of SR-46 was used to conduct the safety evaluation using IHSDM. The project was located on SR-46 about 4 miles of east of US-231 in Indiana as shown in Figure 1. The design exception was proposed to use 11-foot lanes and 2-foot shoulders, rather than the standard 12-foot travel lanes and 6 feet shoulders. A photo of a section of the design exception project is shown in Figure 2.

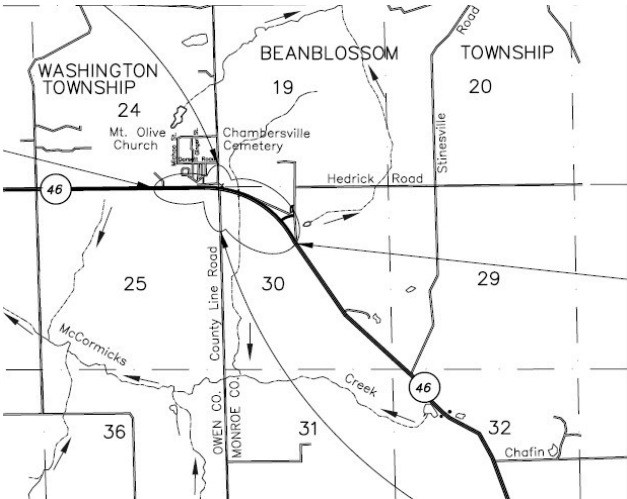


Figure 1. Project Location



Figure 2. A Section of the Design Exception Project on SR 46

The values of the required design elements are given in Table 1. One of the required input values is the roadside hazard rating. A roadside hazard rating is determined in IHSDM according to the roadside safety conditions, ranging from 1 to 7 with 1 being the most favorable roadside safety condition. The roadside hazard rating for this project is rated as 6 based on the following facts: 1). the roadside clear zone is less than five feet; 2). the side slope is approximately 3:1; 3). there are no guardrails, and 4). some rigid obstacles are present within 0 to 6.5 feet from the pavement edge line.

The project was divided into a number of segments in terms of geometric characteristics. The tangent and curve segments and their relevant geometric values are listed in Table 2. The horizontal curve segments are listed in Table 3 along with their radii and lengths. The information in Tables 1, 2 and 3 provides the main input values for IHSDM to perform safety evaluation.

Table 1. Design Elements of the Design Exception Project

Start/End Stations	13+76.970 – 31+41.560
Length (mile)	0.3342
AADT (Measured in 2011)	2400
Project design criteria	3R (Non freeway)
Functional classification	Rural principal arterial
Terrain	Rolling
Design speed	45 mph
Access control	None
Number of lanes & width	2@ 11 feet
Shoulders	2 feet (paved)
Maximum shoulder width	2.05 feet
Minimum shoulder width	0.95 feet
Side slopes	3:1 (6:1 at rock slopes)
Superelevation (%)	2
Roadside Hazard Rating	6.0
Driveway Density (dwys/mi)	26.9

Table 2. Highway Segment Data

Segment	Station		Length (ft)	Lane Width (ft)		Shoulder Width(ft)		Shoulder Type		Driveway Density (dwys/mi)	Roadside Hazard Rating	Grade (%)
	Start	End		Right	Left	Right	Left	Right	Left			
Curve 1	13+76.970	15+07.430	130.46	11.00	11.00	2.0	2.0	paved	paved	26.9	6.0	-6.16
Tangent 1	15+07.430	15+35.830	28.40	11.00	11.00	2.0	2.0	paved	paved	26.9	6.0	-6.16
Curve 2	15+35.830	16+66.300	130.47	11.00	11.00	2.0	2.0	paved	paved	26.9	6.0	-6.16
Tangent 2	16+66.300	18+56.000	189.70	11.00	11.00	2.0	2.0	paved	paved	26.9	6.0	-6.16
Tangent 3	18+56.000	23+94.560	538.56	11.00	11.00	2.0	2.0	paved	paved	26.9	6.0	4.00
Curve 3	23+94.560	28+15.000	420.44	11.00	11.00	2.0	2.0	paved	paved	26.9	6.0	4.00
Curve 4	28+15.000	30+03.420	188.42	11.00	11.00	2.0	2.0	paved	paved	26.9	6.0	-2.29
Curve 5	30+03.420	31+41.560	138.23	11.00	11.00	2.0	2.0	paved	paved	26.9	6.0	-2.29

Table 3. Horizontal Curve Data

Segment	Station		Length of Curve (ft)	Radius (ft)	Superelevation (%)	Design Speed (mph)
	Start	End				
Curve 1	13+76.970	15+07.430	130.46	1150.00	2.0	45
Curve 2	15+35.830	16+66.300	130.47	1150.00	2.0	45
Curve 3	23+94.560	30+03.420	608.86	7669.44	2.0	45
Curve 4	23+94.560	30+03.420	608.86	7669.44	2.0	45
Curve 5	30+03.420	31+41.560	138.23	1650.00	2.0	45

Using the input information described above, the IHSDM Crash Prediction/Accident Analysis Module generated a set of safety evaluation reports. The analysis was specified as a six-year period, 2011 through 2016. The evaluation reports include summaries of the input data as well as the estimated crash frequencies and crash rates. A set of graphs are produced by the software to visually analyze the roadway safety. The graphs illustrate crash rates by segments. The moving average of the crash rates per mile is provided. In addition, roadway elevation and radius are also provided. As shown in the graphs, several segments (from Station 13+76.970 to Station 15+07.430, from 15+35.830 to 16+66.300, and from 30+03.420 to 31+41.560) have relatively high estimated crash rates. It is interesting to note that these segments contain sharper curves. The expected crash summaries in the six-year period are compiled in Tables 4 and 5. Figure 3 plots the expected six-year crashes of the eight segments.

Table 4. Expected Crashes (2011-2016)

Expected Crashes (six years)	
Total Crashes	5.32
Fatal and Injury Crashes	1.71
Fatal and Serious Injury Crashes	0.94
Property-Damage-Only Crashes	3.62
Percent of Total Expected Crashes	
Percent Fatal and Injury Crashes (%)	32
Percent Fatal and Serious Injury Crashes (%)	18
Percent Property-Damage-Only Crashes (%)	68
Expected Crash Rate	
Crash Rate (crashes/mi/yr)	2.6550
Fatal and Injury Crash Rate (crashes/mi/yr)	0.8523
Fatal and Serious Injury Crash Rate (crashes/mi/yr)	0.4673
Property-Damage-Only Crash Rate (crashes/mi/yr)	1.8027

Table 5. Expected Crashes of Roadway Segments

Start Station	End Station	Length (mi)	Expected Crashes	Crash Rate (crashes/mi/yr)
13+76.970	15+07.430	0.0247	0.92	6.1772
15+07.430	15+35.830	0.0054	0.06	1.8008
15+35.830	16+66.300	0.0247	0.92	6.1769
16+66.300	18+56.000	0.0376	0.41	1.8008
18+56.000	23+94.560	0.1003	1.03	1.7076
23+94.560	28+15.000	0.0796	0.97	2.0245
28+15.000	30+03.420	0.0357	0.39	1.8404
30+03.420	31+41.560	0.0262	0.64	4.0684

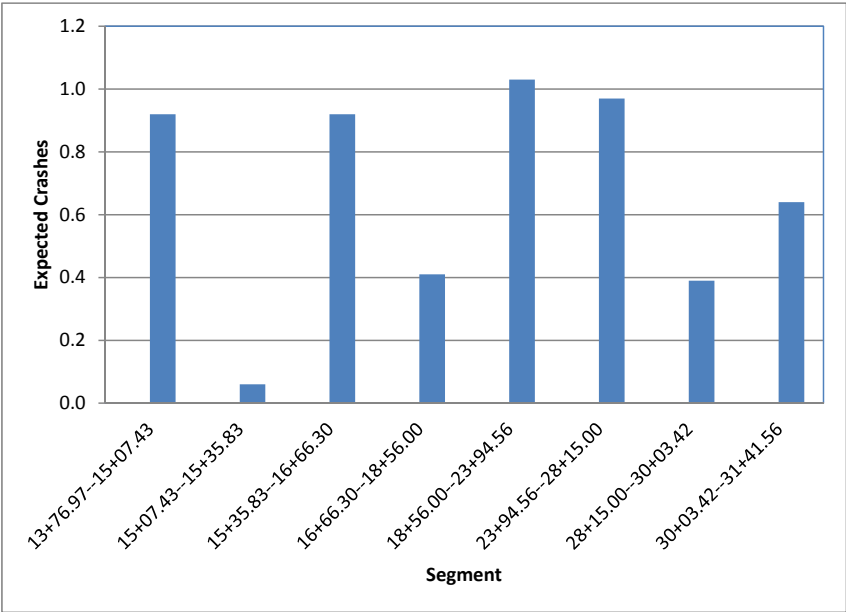


Figure 3. Expected Crashes of the Roadway Segments

BENEFIT-COST ANALYSIS

The FHWA publication, “Economic Analysis Primer” (FHWA, 2006), is a great source of economic analysis methods for highway projects. The FHWA publication explains the differences between LCCA and BCA methods and the appropriate applications of them as follows. LCCA is applied when an agency must undertake a project and is seeking to determine the lowest life-cycle-cost (i.e., most cost-effective) means to accomplish the project’s objectives. LCCA enables the analyst to make sure that the selection of a design alternative is not based solely on the lowest initial costs, but also considers all the future costs (appropriately discounted) over the project’s usable life. LCCA is used appropriately only to select from among design alternatives that would yield the same level of performance or benefits to the project’s users during normal operations. If benefits vary among the design alternatives (e.g., they would accommodate different levels of traffic), then the alternatives cannot be compared solely on the basis of cost. Rather, the analyst would need to employ benefit-cost analysis (BCA), which measures the monetary value of life-cycle benefits as well as costs. Accordingly, LCCA should be viewed as a distinct, cost-only subset of BCA. The BCA process begins with the establishment of objectives for a highway project and development a set of alternatives for evaluation. For applications of BCA in design exceptions, the alternatives can be a design that would meet the standard and the proposed design exception options. BCA can evaluate and compare several alternatives, so it can include more than one proposed design exception options in the evaluation. To ensure that the alternatives can be compared fairly, the analyst specifies

a multiyear analysis period over which the life-cycle costs and benefits of all alternatives will be measured. The investment costs, hours of delay, crash rates, and other effects of each alternative are measured. The analyst assigns dollar values to the different effects and discounts them to a present value amount. Risk associated with uncertain costs, traffic levels, and economic values also is assessed.

The Economic Analysis Primer (FHWA, 2006) describes the two most common measures to compare benefits to costs in BCA below.

- Net present value (NPV). NPV is perhaps the most straightforward BCA measure. All benefits and costs over an alternative’s life cycle are discounted to the present, and the costs are subtracted from the benefits to yield a NPV. If benefits exceed costs, the NPV is positive and the project is worth pursuing. Where two or more alternatives for a project exist, the one with the highest NPV over an equivalent analysis period should usually be pursued. Policy issues, perceived risk, and funding availability, however, may lead to the selection of an alternative with a lower, positive NPV.
- Benefit-cost ratio (BCR): The BCR is frequently used to select among projects when funding restrictions apply. In this measure, the present value of benefits (including negative benefits) is placed in the numerator of the ratio and the present value of the initial agency investment cost is placed in the denominator. The ratio is usually expressed as a quotient. For any given budget, the projects with the highest BCRs can be selected to form a package of projects that yields the greatest multiple of benefits to costs. FHWA recommends that only the initial agency investment cost be included in the denominator of the ratio. All other BCA values, including periodic rehabilitation costs or user costs, such as delay associated with construction, should be included in the ratio’s numerator as positive or negative benefits. Adherence to this guidance facilitates consistent project comparisons.

Based on the review of the INDOT design exception projects, it is determined that the cost and benefit items shown in Table 6 should be included in the BCA. Agency costs are estimated by engineers or designers based on past experience, bid prices, design specifications, materials costs, and other information. Although land acquisition is not usually involved in INDOT’s design exception projects, it is included in the agency costs in case additional land is needed to meet the standard design requirements.

Table 6. Benefits and Costs for Design Exception Projects

Agency Costs
Design and engineering
Land acquisition
Construction
User Costs/Benefits Associated with Highway Operations
Travel time
Vehicle operating costs
Crashes

Travel time and delay costs are usually valued as a percentage of average personal wages. The U.S. Department of Transportation (USDOT) provides the average wages of different categories of travelers and recommends the percents of average wages for economic analysis (USDOT, 2011) shown in Table 7. The value of reduced travel time often accounts for the greatest share of a transportation project’s benefits.

Table 7. Recommended Values of Travel Time Savings

Category	Purpose of Travel	Hourly Earnings Rates for Values of Travel Time Savings (2009 U.S. \$ per person-hour)	% of Earning for Economic Evaluation
Local Travel	Personal	\$23.9	50%
	Business	\$22.9	100%
Intercity Travel	Personal	\$23.9	70%
	Business	\$22.9	100%

Crash costs are the monetary values for fatalities and injuries associated with crashes. The crash cost values used by different agencies vary significantly. Table 8 shows the crash cost values from INDOT (Gkritza et al., 2006) and FHWA (2010b). The cost values in Table 8 indicate that the use of crash costs from different agencies may make considerable differences in the benefit-cost analysis.

Table 8. INDOT and FHWA Recommended Crash Costs

Crash Severity	Indiana	FHWA
Fatal/Injury	\$81,866	\$158,200
Property Damage Only	\$6,822	\$7,400

The vehicle operating costs can be affected by a highway project due to the changes that it causes in highway speeds, traffic congestion, pavement surface, and other conditions that affect vehicle fuel consumption and wear and tear. The AASHTO publication, User and Non-User Benefit Analysis for Highways (AASHTO, 2010b), provides good information on the valuation of vehicle operating costs (VOC).

Traffic volumes affect greatly the results of benefit and cost of a highway project in terms of user benefits and vehicle operating cost savings. Therefore, accurate measurements and forecasts of traffic volumes are critical to obtaining valid results from BCA. The most commonly used method for predicting the future traffic volumes is the use of an annual growth rate of traffic. It should be pointed out that use of a fixed growth rate of traffic after a highway project is constructed may not be realistic in many cases.

In order to provide an efficient tool for conducting benefit-cost analysis for design exception projects, an Excel based computer program was developed. With this computer program, engineers are able to conduct benefit-cost analysis efficiently and

accurately for design exception projects. In the benefit analysis, the design exception alternatives are compared to the design that meets all the design standards. In addition to the initial costs, the user benefits or costs of the roadway section during the service period are also considered in the benefit-cost analysis.

To analyze and compare the benefits and costs of the standard design and the design exception, the standard design is used as the base and the design exception is used as an alternative. The initial costs and the user benefits/costs of the base and the alternative should be estimated in the benefit-cost analysis. The initial costs of a project include engineering cost, right-of-way cost, and construction cost. The difference between the initial costs of the standard design project and the initial costs of the design exception project is estimated as \$243,250. That is, if the project is built based on the proposed design exception, it will save the agency an initial cost of \$243,250 in comparison with the standard design. Therefore, the design exception would save \$243,250 in terms of initial costs. This saving is called agency benefit.

In addition to the agency benefit, the user benefits associated with highway operations after completion of the construction should also be considered in the benefit-cost analysis. These user benefits include the monetary values of travel time costs, vehicle operating costs, and crash costs. They are estimated by calculating the differences in the costs between the standard design and the design exception. When the difference in costs is positive, it means the design exception alternative reduces cost and thus a benefit is realized. If the difference in costs is negative, it means the design exception increases cost in comparison with the standard design. A negative user benefit can be considered a user cost. Since user benefits and costs are incurred annually, they are often converted to the present value with a discount rate to compare all the cost and benefit items in the same base year. The net present value (NPV) of the total benefit of a proposed design exception project can be expressed as:

$$NPV =$$

$$\text{Agency Benefit} + \text{Travel Time Savings} + \text{Vehicle Operating Savings} + \text{Crash Savings} \quad (3)$$

It should be noted that values of the user savings in Equation 3 are often negative because they are related to the operations of the highway that were designed with lower criteria than the standard specifications. It is therefore expected that NPV will decrease as time increases.

Agency Benefit: The agency benefit for a design exception project can be calculated as:

$$\text{Agency Benefit} = (\text{Initial cost of standard design}) - (\text{Initial cost of design exception}) \quad (4)$$

Travel Time Cost Savings: Travel time costs are directly related to the vehicle speeds as well as traffic flow rates on the roadway section. The difference of the costs between the standard design alternative and the design exception alternative is the user benefit for using the roadway with the design exception. The average travel speeds on the standard design roadway and on the design exception roadway can be calculated using the method from the Highway Capacity Manual 2010 [10]. Since lane widths and shoulder widths affect the average travel speeds, the standard design and design exception will result in different average travel speeds. The steps for calculating travel time savings are presented below:

Step 1: Predicting traffic volume based on the current traffic volume:

$$AADT_t = AADT_0(1 + r)^t \quad (5)$$

Where,

$AADT_t$ = Average Annual Daily Traffic in Year t ;

$AADT_0$ = Average Annual Daily Traffic in Year 0;

r = annual traffic volume growth rate.

Step 2: Determining hourly traffic flow rate for each vehicle class:

Hourly Traffic Flow Rate = $AADT \times (\text{Hourly Traffic Distribution Factor})$ (6)

Step 3: Determining average speed for each vehicle class (TRB, 2010):

$$\text{Average Speed} = \frac{\text{Free Flow Speed}}{(1 + 0.15 \times (\frac{V}{C})^4)} \quad (7)$$

Where:

V = Hourly traffic volume or flow rate;

C = Highway capacity, the maximum number of vehicles that can pass a given section during a given period of time under prevailing roadway, traffic and control condition.

Step 4: Calculating travel time for vehicles to traverse the given section of roadway.

$$\text{Travel Time} = \frac{\text{Roadway Length}}{\text{Average Speed}} \quad (8)$$

Step 5: Calculating hourly travel time savings of the design exception in comparison with the standard design:

$$\text{HPT} = \text{Hourly Traffic Volume} \times \text{AVO} \quad (9)$$

$$\text{HTTS} = \frac{1}{2} \times (\text{TT}_{\text{SD}} - \text{TT}_{\text{DE}}) \times (\text{HPT}_{\text{SD}} + \text{HPT}_{\text{DE}}) \times \text{VT} \quad (10)$$

Where:

HPT = Hourly Person Trips;

AVO = Average Vehicle Occupancy;

HTTS = Hourly Travel Time Savings;

TT = Travel Time;

DE = Design Exception;

SD = Standard Design;

VT = Value of time (\$/person).

Step 6: Calculating yearly travel time savings (year 1 and the last year of the analysis period):

$$\text{Yearly TT Savings} = \sum_{i=1}^{24} \text{Hourly TT Savings}_i \times 365 \quad (11)$$

Step 7: Calculating intermediate yearly travel time savings by interpolation:

$$\text{Year } m \text{ TT Savings} = \frac{\text{YTTS}_N - \text{YTTS}_1}{AP - 1} \times (m - 1) + \text{YTTS}_1 \quad (12)$$

• Where:

• YTTS = Yearly Travel Time Savings;

• AP = Analysis Period;

• N = the last year of the analysis period;

• m = the m^{th} year within the analysis period.

Step 8: Converting each year's travel time savings into present money value:

$$P = F_n \times \frac{1}{(1+i)^n} \quad (13)$$

Where,

P = Present Value;

F_n = Value in the n^{th} year;

i = Interest Rate/Discount Rate.

Vehicle Operating Cost Savings: Similar to travel time costs, vehicle operating costs are also directly related to the vehicle speeds as well as traffic flow rates on the roadway section. The difference of the vehicle operating costs between the standard design alternative and the design exception alternative is the user benefit for using the roadway with the design exception. The procedures for estimating vehicle operating costs are as follows:

Step 1: Determining hourly fuel cost for each vehicle class:

$$\text{VMT} = \text{AADT} \times \text{Hourly Traffic Distribution} \times L \quad (14)$$

Where:

VMT = Hourly vehicle-miles traveled;

L = Length of the roadway section.

$$\text{Hourly Fuel Cost} = \text{VMT} \times \text{Fuel Consumption Rate} \times \text{Unit Fuel Cost} \quad (15)$$

Step 2: Determining truck inventory cost:

$$\text{Hourly Truck Inventory Cost} = \text{VMT} \times \text{Cargo Value} \times \frac{\text{Interest Rate}}{365 \times 24} \times \left(\frac{1}{\text{Average Speed}} \right) \quad (16)$$

Step 3: Determining hourly total VOC and VOC savings:

$$\text{Hourly Total VOC} = \text{Hourly Fuel Cost} \times \text{VOC factor} + \text{Hourly Truck Inventory Cost} \quad (17)$$

Where:

VOC factor = $1/0.7 = 1.43$, assuming fuel costs account for 70% of total VOC.

$$\text{Hourly VOC Savings} = \text{Hourly Total VOC}_{\text{SD}} - \text{Hourly Total VOC}_{\text{DE}} \quad (18)$$

Where:

DE = Design Exception;

SD = Standard Design;

Step 4: Calculating yearly VOC savings (year 1 and the last year of the analysis period):

$$\text{Yearly VOC Savings} = \sum_{i=1}^{24} \text{Hourly VOC Savings}_i \times 365 \quad (19)$$

Step 5: Calculating intermediate yearly travel time savings by interpolation:

$$\text{Year } m \text{ VOC Savings} = \frac{\text{YVOCS}_N - \text{YVOCS}_1}{AP - 1} \times (m - 1) + \text{YVOCS}_1 \quad (20)$$

Where:

YVOCS = Yearly VOC Savings;

AP = Analysis Period;

N = the last year of the analysis period;

m = the m^{th} year within the analysis period.

Step 6: Converting each year's VOC savings into present money value using Equation 13:

$$P = F_n \times \frac{1}{(1+i)^n} \quad (21)$$

Crash Cost Savings: The difference of the crash costs between the standard design alternative and the design exception alternative is the user benefit for using the roadway with the design exception. As Table 8 shows, INDOT and FHWA recommend different values of the costs of different crashes. In this application example, both INDOT and FHWA crash cost values are used in the benefit-cost analysis for comparison purpose.

Benefit-Cost Analysis Results: Following the methods discussed above, all the benefits and costs can be computed with the Excel based computer program. In this benefit-cost analysis, a discount rate of money of 4% and an annual traffic growth rate of 4% are used. The standard design requires a lane width of 12 feet and a shoulder width of 6 feet. The design exception option proposes a lane width of 11 feet and a shoulder width of 2 feet and use of additional speed limit and warning signs. As

Equation 3 shows, the net present value of total benefit is the summation of all benefits or savings:

$$NPV = \text{Agency Benefit} + \text{Travel Time Savings} + \text{Vehicle Operating Savings} + \text{Crash Savings}$$

Using the current year as the base year and a service life of 20 years, the costs of the proposed design exception are compared to those of the standard design. If the difference between a cost of the standard design and a corresponding cost of the design exception is positive, it indicates that the design exception has a positive benefit. Otherwise, the design exception has a negative benefit. A negative benefit is actually a cost resulted from adoption of the design exception instead of the standard design.

The agency benefit in this example is given as:

$$\text{Agency Benefit} = (\text{Initial cost of standard design}) - (\text{Initial cost of design exception}) = \$243,250$$

That is, implementing the design exception project will save \$243,250 of initial costs in terms of costs of design and engineering, land acquisition, and construction in comparison with the standard design option. Since this agency benefit incurs in the base year (year 0), the benefit is already the present value and there is no need to convert the value with discount rate.

With the Excel based computer program, the travel time cost savings, the vehicle operating cost savings, and the crash cost savings were calculated for a 20-year service life. Each of the cost savings is the difference between the corresponding costs of the standard design and the design exception. Table 9 lists the user benefit values of the design exception project in each of the 20 years of the service life. All the user benefit values for each year in Table 9 are expressed in present values, i.e., in the current (2011) dollar values. The crash cost savings were calculated based on the INDOT crash cost values as well as on the FHWA crash cost values listed in Table 8. The values of user benefits listed in Table 9 are all negative. This indicates that, compared to the standard design, the design exception will cost users more in terms of longer travel time, higher vehicle operating expenses, and more estimated vehicle crashes.

The total benefits of the proposed design exception project can be calculated with Equation 3 using the agency benefit of \$243,250 and the user benefits listed in Table 9. The calculated total benefits are presented in Table 10. As can be seen from this table, in the beginning of the analysis period, Year 0, the total benefit is nothing but the agency benefit. That is, compared to the standard design, the design exception option would initially save \$243,250 in terms of engineering, land acquisition, and construction costs. Because of the negative user benefits, the total benefit will decrease each year as motorists will endure negative impact of the roadway with design exceptions. As service time increases, the total benefit will eventually decrease from positive to negative. Therefore, use of the savings in initial costs alone to justify a design exception project may not be appropriate as this application example indicates.

To demonstrate the trends of the total benefits, Figure 4 displays the total benefits of the design exception project in different years. With the INDOT recommended crash cost values, the total benefit will become negative in the 10th service year. If the FHWA recommended crash cost values are used, the total benefit will change to negative in the 9th service year. Therefore, if the expected service life is longer than 10 years, the design exception project will not be justified in terms of the total benefit.

Table 9. User Benefits of the Design Exception Project (Present Values)

Year	Travel Time Savings	Vehicle Operating Cost Savings	Crash Cost Savings	
			Based on INDOT Crash Cost Values	Based on FHWA Crash Cost Values
2012	-\$19,261.58	-\$1,777.03	-\$3,532.17	-\$6,397.93
2013	-\$19,599.68	-\$1,808.22	-\$3,396.32	-\$6,151.85
2014	-\$19,883.28	-\$1,834.39	-\$3,382.32	-\$6,041.76
2015	-\$20,116.07	-\$1,855.86	-\$3,140.09	-\$5,687.74
2016	-\$20,301.54	-\$1,872.97	-\$3,019.32	-\$5,468.98
2017	-\$20,442.99	-\$1,886.02	-\$2,955.03	-\$5,314.87
2018	-\$20,543.52	-\$1,895.30	-\$2,841.38	-\$5,110.45
2019	-\$20,606.08	-\$1,901.07	-\$2,684.16	-\$4,861.90
2020	-\$20,633.44	-\$1,903.59	-\$2,580.93	-\$4,674.90
2021	-\$20,628.21	-\$1,903.11	-\$2,525.97	-\$4,543.17
2022	-\$20,592.86	-\$1,899.85	-\$2,386.21	-\$4,322.21
2023	-\$20,529.71	-\$1,894.02	-\$2,294.43	-\$4,155.97
2024	-\$20,440.96	-\$1,885.84	-\$2,284.98	-\$4,081.59
2025	-\$20,328.66	-\$1,875.48	-\$2,121.33	-\$3,842.43
2026	-\$20,194.77	-\$1,863.12	-\$2,039.74	-\$3,694.64
2027	-\$20,041.10	-\$1,848.95	-\$1,961.29	-\$3,552.54
2028	-\$19,869.38	-\$1,833.10	-\$1,919.53	-\$3,452.44
2029	-\$19,681.23	-\$1,815.74	-\$1,845.70	-\$3,319.65
2030	-\$19,478.15	-\$1,797.01	-\$1,743.58	-\$3,158.20
2031	-\$19,261.58	-\$1,777.03	-\$1,676.52	-\$3,036.73

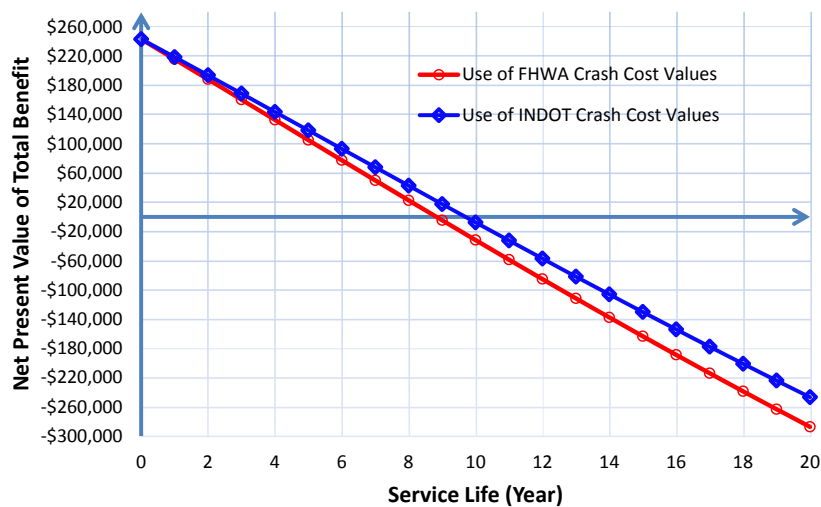


Figure 4. Total Benefits in Different Service Years

Table 10. Total Benefits of the Design Exception Project

Calendar Year	Service Year	Net Present Value of Total Benefit	
		Based on INDOT Crash Cost Values	Based on FHWA Crash Cost Values
2011	0	\$243,250	\$243,250
2012	1	\$218,679	\$215,813
2013	2	\$193,875	\$188,254
2014	3	\$168,775	\$160,494
2015	4	\$143,663	\$132,835
2016	5	\$118,469	\$105,191
2017	6	\$93,185	\$77,547
2018	7	\$67,905	\$49,998
2019	8	\$42,714	\$22,629
2020	9	\$17,596	-\$4,583
2021	10	-\$7,462	-\$31,657
2022	11	-\$32,341	-\$58,472
2023	12	-\$57,059	-\$85,052
2024	13	-\$81,670	-\$111,460
2025	14	-\$105,996	-\$137,507
2026	15	-\$130,094	-\$163,260
2027	16	-\$153,945	-\$188,702
2028	17	-\$177,567	-\$213,857
2029	18	-\$200,910	-\$238,674
2030	19	-\$223,928	-\$263,107
2031	20	-\$246,643	-\$287,182

CONCLUSIONS

A thorough evaluation of IHSDM and HSM was conducted to explore the feasibility of using the tools for safety assessment of design exception projects. A case study was performed to illustrate the process of safety evaluation. It was demonstrated that IHSDM can be used to generate quantitative measures of safety impacts of design exception projects. IHSDM is capable of analyzing safety impacts of an individual substandard element as well as combined effects of a number of substandard elements. With IHSDM, the sensitivity of substandard elements can be analyzed by changing the values of design criteria. Using different combinations of substandard elements, such as lane width and shoulder width combinations, designers can choose the best alternative that would minimize the negative safety impacts.

One of the commonly used methods for justifying design exception projects is to use the savings in construction cost. However, this method is not a reasonable one because it does not include the impacts of a substandard highway section to the highway safety and operations. In this study, benefit-cost analysis method was used to evaluate the effectiveness of design exceptions. An Excel based computer program was developed to conduct benefit-cost analysis for design exceptions. This method includes not only the savings in construction cost and other initial costs, but also the user

benefits in terms of travel time, vehicle operation, and safety. The computer program provides a useful and convenient tool for highway engineers and planners to evaluate design exception projects.

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Convective Depth of the Chloride Ion in Concrete Surfaces

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ABSTRACT

Based on the hypothesis of chloride ion concentration in concrete surfaces changing linearly, a convective depth model was established in this study. The rationality of the model was verified by experiment. In addition, a novel method for calculating the equivalent chloride ion content in the natural environment is proposed. The results showed that the diffusion of the chloride ion in concrete can be described by Fick's second law, and there was an exact convective depth value in a concrete surface. In general, the value of the convective depth is relative to the wetting-drying ratio of the environment and the permeability coefficient of the concrete. Compared with the water influence depth in concrete, the convective depth was shallow. Moreover, the equivalent chloride ion content in the environment was determined by the characteristics of the environment.

Key Words: Concrete; convective depth; diffusivity; chloride ion

INTRODUCTION

It is well known that the chloride ion is the main factor inducing the deterioration of concrete structures in marine environments. Much research about this has been carried out at home and abroad, and many research results have been obtained. Although the optimal chloride diffusion model is Fick's second law, it still has some shortcomings. For example, the convective depth of the chloride ion is usually obtained by fitting the chloride diffusion profile, which is the indirect method with testing error (Ann 2009; Zhang 1998). How to ensure the chloride convective depth and the concentration in concrete surfaces is obviously significant for forecasting service life and evaluating the durability of reinforced concrete structures in marine environments.

Most existing research on chloride convective depth in concrete surfaces was based on measured data, and the fitting curve of the data was used to obtain

the value. Some standards regarded the chloride convective depth value as a constant value, such as 14mm recommended by the Duracrete standard (BE95-1347 2000). But the designated value was so simple that it was not suitable for special conditions. In addition, some researchers set the value as 20mm (Rincon 2004). Vesikari (2003) revealed that the chloride convective depth in concrete approximately accorded with the Beta function by studying the chloride profiles of more than 127 typical engineering situations. Fan et al. (2006) indicated that the chloride convective depth in concrete was about 15mm by discussing the chloride curve of the port in marine environments. Li (2009) regarded the water influence depth as the chloride convective depth in concrete. Generally speaking, the chloride convective depth in concrete depends on the environment, the drying-wetting time ratio, load, and the properties of the concrete (Michael 1999). However, most of time, the research referred to above does not pay attention to these situations, which raises many questions. Furthermore, the test methods also influenced the precision of the convective depth.

In this study, the change law of the chloride ion in concrete under natural and artificial environments was investigated. Based on the hypothesis of the chloride ion concentration in concrete cover changing in linearly, the convective depth model was obtained. In addition, the rationality of the model was verified by testing.

THEORETICAL MODEL

Based on the distribution law of the chloride ion in concrete under drying-wetting environments, concrete can be divided into two parts—convective zone and internal diffusion zone (Castro 1997). The author considered the distribution of the chloride associated with the drying-wetting action and pore structure of the concrete, and the chloride transmission mode within convective zone was still by diffusion. Based on the hypothesis of chloride diffusion in semi-infinite concrete processing along the direction of one dimension and the concentration gradient changing along the convective zone to the steel surface, Fick's second law, which was a function of time and space, was used to describe the diffusion, as shown in Equation (1).

$$C(x,t) = C_0 + (C_s - C_0) \left[1 - \operatorname{erf} \left(\frac{x - \Delta x}{2\sqrt{D_{app}t}} \right) \right] \quad (1)$$

Where $C(x,t)$ is the chloride content in concrete at depth x and exposure time t , %; C_s is the chloride content at the concrete surface, %; C_0 is the chloride content at the initial period, %; t is the exposure time, s; x is the distance from the concrete surface, m; D_{app} is the diffusion coefficient, m^2/s ; Δx is the convective depth in concrete, m; and $\operatorname{erf}(z)$ is the error function.

From Equation (1), it can be seen that ensuring the value of convective depth Δx and concrete surface chloride content C_s was very significant for

forecasting service life and evaluating the durability of reinforced concrete structures. The accumulation of chloride in alternating wetting-drying convective zones was very complex and, most of time, was the coupling effect of multiple factors, including convection and diffusion. Some researchers revealed that the concentration of chloride in a convective zone was a constant value, and the corresponding depth was regarded as the convective depth in concrete. Most of existing studies about the value of Δx were based on measured data and deduced by Equation (1), which still faced some controversy, such as the subjective evaluation had poor transparency and reappearance, the objective method lacks a theoretical foundation, and the subjective and objective weights tended to pay no attention to theory. If the diffusion coefficient D_{app} , initial chloride concentration C_0 , and concrete surface chloride content C_s were all regarded as constant values, then the convective depth Δx can be indirectly obtained by Equation (1) and as shown in Equation (2).

$$\frac{C(x, t_1) - C(x, t_2)}{C_s - C_0} = \operatorname{erf}\left(\frac{x - \Delta x}{2\sqrt{D_{app}t_2}}\right) - \operatorname{erf}\left(\frac{x - \Delta x}{2\sqrt{D_{app}t_1}}\right) \quad (2)$$

Although the method above can obtain the convective depth value Δx , there are some shortcomings, including being time-consuming and achieving low accuracy. Moreover, it cannot directly obtain the convective depth value Δx in the absence of the concrete's physical characteristics and environment. In other words, it cannot overcome long-time field testing.

Li (2009) recommended water influence depth as the convective depth of chloride in concrete, which did not match with the facts. At present, accurately obtaining the convective depth is rarely of concern at home and abroad.

Based on the hypothesis of chloride concentration with linearity in the convective zone and referencing Li (2009), the model of the convective depth in concrete cover was inferred. This study was conducted as follows: the chloride salt solution was a chloride ion-containing solution that was absorbed by capillary suction of unsaturated concrete under continuous wetting-drying cycles. In the early drying process, the chlorine salt solution was transferred bi-directionally into the concrete inner and surface. In the early drying stages, the water transferred from concrete inner in the form of vapor under the condition of pore water without sustaining continuous status, and the chloride salt concentrated and crystallized in the concrete. Generally speaking, the chloride concentration in the convective zone maximized and maintained balance with time. Due to hysteresis and the blocking effects of the concrete, the evaporation and condensation effects occur only on the concrete surface. Therefore, the chloride convective depth was less than the water influence depth. When the concrete was wet again, the chloride salt solution was absorbed by capillary suction of the unsaturated concrete, and the chloride ion was absorbed into the concrete and supplemented the consumption. Concentration of the chloride at a certain depth in the concrete reached a new equilibrium state with the drying-wetting test, and the corresponding convective depth and limited chloride concentration were

generated, which was related to the drying-wetting time ratio. In this study, the convective depth in concrete was established, as shown in Figure1. The corresponding water influence curve is shown in the upper portion of Figure 1, and the corresponding convective depth model profile is described below.

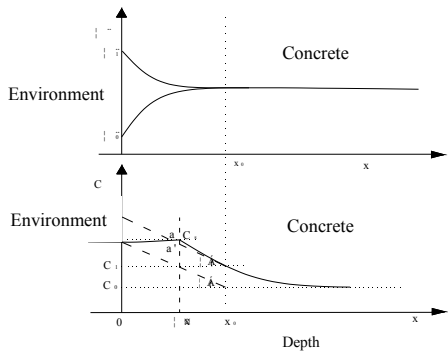


Figure 1. Convective depth model of chloride ion in concrete surface.

In this study, the hypothesis of chloride concentration in water influence depth zone x_0 was regarded as fitting to a linear function. The equivalent chloride concentration of environment C_e was perceived as a constant, and Fick's second law was used to describe the distribution of the chloride within the convective depth zone. Concentrations of the chloride at the water influence depth x_0 and convective depth Δx were expressed as C_1 and C_s , respectively. From Figure 1, it can be seen that the chloride ingress profile can be represented as Equation (3) based on the hypothesis of the linear function, shown as dotted line in Figure 1, and the corresponding model of the convective depth Δx can be shown as Equation (4).

$$\operatorname{tg} \alpha=\frac{C_e-C_0}{x_0}=\frac{C_s-C_1}{x_0-\Delta x} \quad (3)$$

$$\Delta x=\left(1-\frac{C_s-C_1}{C_e-C_0}\right) x_0 \quad (4)$$

As far as is known, the initial chloride concentration C_0 , the equivalent chloride concentration of the environment C_e , and the water influence depth C_1 can be directly measured in concrete. Moreover, the concrete surface chloride concentration C_s can also be calculated, which is discussed below. The corresponding depth x_0 based on the model of the water influence depth by Li (2009) can be obtained, and the convective depth Δx can be calculated by Equation (4). From Equation (4), it is seen that the convective depth Δx is the function of the water influence depth, which overcame the shortcomings of the traditional methods.

EXPERIMENT PROCESS

Grade 42.5 Portland cement, polycarboxylic type high performance water reducer, class I fly ash, grade S95 slag, water, sand, and limestone the size of 5–20 mm were used as the main raw materials to produce concrete. The mass ratio of the limestone, sand, cement, water, slag, fly ash, and water reducer was 1060:710:355:155:70:45:4.5, and the sizes of the samples were 150 mm×150 mm×150 mm and 150 mm×150 mm×400 mm. Only one side of the samples was left for testing; the other sides were sealed with epoxy. There were three stages for an experiment cycle, and the cycling time was about 72 hours. A spray stage was set for 50 minutes, and the last period was for drying. Temperature and duration time were set as 40°C/30 h, 50°C/12 h, and 60°C/30 h. The concentration of the salt solution was 5%, and the wind speed was set as 3m/s. A profile grinding machine (Model PF1100 produced in Denmark) was used to prepare the powder; its fineness passed the sieve with the size of 75 μm . Then, the chloride content in the concrete was tested according to China standard JTJ 270-1998. The profile grinding machine and samples are shown in Figure 2.

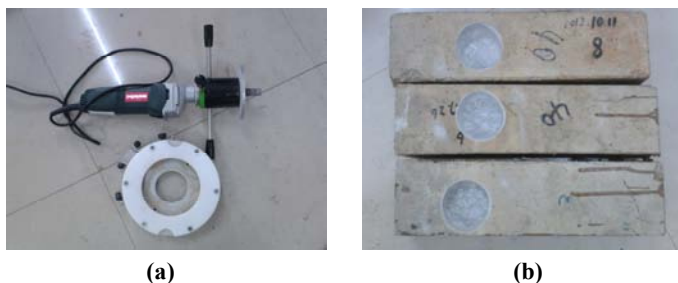


Figure 2. Profile grinding machine and sample.

RESULTS AND DISCUSSION

To verify the rationality of the chloride convective depth model in concrete surfaces, an artificial simulation environment test was carried out. The initial water saturation in the concrete was 0.8, and the environment water saturation applied to the concrete was set as 0.4. The initial chloride concentration of the concrete was 0.016%, which was the mass of the concrete. Figure 3 shows the curve of the water saturation distribution of the concrete in the simulation environment, and the chloride ion content profile in concrete for eight months under artificial simulation environment is shown in Figure 4.

From Figure 3, it can be seen that the water saturation on the concrete surface changed only in a certain depth zone x_0 , and its value did not change when the value was more than a certain depth. The water influence depth x_0 was about 14mm, which was calculated based on Li (2009). The following sections investigate the simulation test results used to calculate the chloride convective depth Δx in concrete.

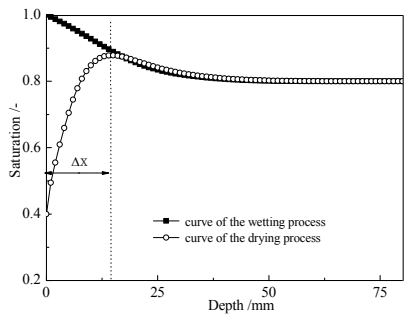


Figure 3. Water saturation distribution curve of concrete in simulation environment.

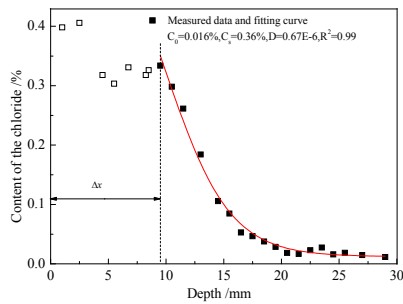


Figure 4. Chloride ion content curve of concrete in artificial simulation environment.

It can be seen from Figure 4 that the chloride concentration can be described by Fick’s second law when the chloride convective depth Δx was more than a certain value and the measured data perfectly fit the fitted curve. The chloride convective depth Δx calculated by the Equation (1) was about 10 mm. Although the method above could be used to calculate the chloride convective depth, it also had some shortcomings, including lengthy time consumption, measured data, and low precision. To overcome these difficulties, a chloride convective depth model in the concrete based on the special environment and property of the concrete was established in this study. Among the influences of the theoretical simulation factors, the water influence depth x_0 was estimated by numerical simulation. In view of the results above, the corresponding chloride convective depth of the concrete in the artificial simulation environment was about 11mm, and the value was very close to the measured value. In addition, the chloride convective depth of a trestle bridge built for 12 years in a marine environment was also calculated. The tidal range infiltrated time ratio was about 0.776, and the sea water salt concentration of the test point was about 2.9%,

regarded as the chloride salt value. The water influence depth of the concrete structure was about 6mm, which was calculated by theoretical simulation. Figure 5 shows the curve of the chloride ion content (mass of concrete) of the trestle bridge.

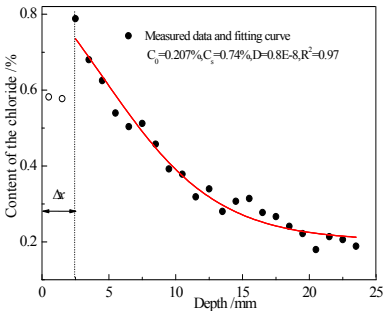


Figure 5. Chloride ion content curve of trestle bridge.

Figure 5 reveals that the chloride concentration in concrete can also be described by Fick’s second law, when the depth was more than a certain value. The measured data perfectly accorded with the fitting curve, and the chloride concentration in a certain depth of the concrete was a constant value. From the results discussed above, it can be seen that the convective depth also existed in field concrete structures in a marine environment. The chloride convective depth of the concrete structure was about 3.5mm, and the measured data was about 2.5mm, which was in accordance with the calculation. It also indicated that the model used to calculate the chloride convective depth of the structure in marine environments was feasible.

CONCLUSIONS

From the discussion above, conclusions can be made as follows:

- (1) Distribution of chloride ion content in concrete in drying-wetting environments indicated that the concrete can be divided into two parts - a convective zone and an internal diffusion zone, and the distribution of chloride ion within the convective zone can be described by Fick’s second law.
- (2) Based on the linear function hypothesis of the chloride content change in the convective zone, a chloride convective depth model was established. By analyzing the measured data of the concrete in an artificial simulation environment and a marine environment, the rationality of the model was proved.

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Research on Ingress of Chloride Ions in Concrete in an Artificial Simulation Environment

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ABSTRACT

The time-varying of the diffusion coefficient and distribution of the chloride ion content in concrete was investigated in this paper. Based on the testing time and methods, the difference and the application range of the diffusion coefficient are discussed. In addition, the difference of the chloride ion ingress in concrete with various strength levels was also researched. Results showed that chloride diffusion in concrete can be described by Fick's diffusion law, and the fitting curve based on the annual diffusion coefficient was more suitable for describing the change. Generally speaking, the diffusion coefficient tends to stabilize the value when the testing time is more than a certain constant value.

Key Words: Chloride ion; artificial simulation environment; time-varying

THEORETICAL MODEL

Time-varying law of chloride in concrete surfaces. At present, Fick's second law is generally used to describe chloride distribution in concrete, and the chloride concentration of concrete surfaces is regarded as a constant value (Tang 1999; Troconis 2004). However, measured data show that the change in chloride concentration of concrete surfaces is an accumulated process with time, and it is a constant value. Generally speaking, there are usually many models for expressing the chloride concentration of concrete—linear, multinomial, power function, square root, exponential function, etc. (Amey 1998; Bamforth 1999; Weyers 1998; Song 2008). Although research indicates that the results perfectly accord with measured data, the adaptability and correlation of the models remain to be investigated. For example, most of research employs Fick's second law and constant boundary conditions to determine the chloride concentration, and the fitting curve is used to discuss the time variance of chloride concentration, which was contrary to logic. Although DuraCrete regards the chloride concentration of concrete surfaces as a function of water to cementitious materials, listed as

Equation (1), some studies have revealed that the chloride concentration with large water to cementitious materials was low, which may be due to the skin effect. Moreover, the effect of the environment and materials on chloride concentration time-varying was rarely investigated (Gao 2008).

$$C_s = A(W/C) + \varepsilon \quad (1)$$

Where A and ε are the fitted values and W/C is water to cementitious materials.

From the discussion above, a modified exponential function model was adopted to describe the time-varying of the chloride concentration in concrete (Jiang 2011), shown as Equation (2).

$$C_s(t) = C_0 + C_{\max}(1 - e^{-rt}) \quad (2)$$

Where $C_s(t)$ is the chloride concentration at time t (mass of concrete), %; $C_{\max}$ is the chloride concentration in stable status, %; C_0 is the initial chloride concentration in concrete, %; and the r is the fitted coefficient.

Correlation between free chloride and chloride binding. A study revealed that the chloride concentration of the solution could affect adsorption and the distribution of the chloride ion in concrete. Because the solution concentration was different for the artificial simulation environment and the natural environment, it was necessary to discuss its influence on the chloride concentration of the concrete surface. To set optimal chloride concentration of the solution, the correlation between free chloride and chloride binding was investigated. There are two kinds of the chloride ions in concrete—free chloride and chloride binding (Thomas 1999).

The binding coefficient of chloride was different, which was due to the binding mechanism of the models. So the linear binding mechanism of Freundlich and Langmuir was investigated as follows (Qiao 2012). Equation (3) gives the linear binding mechanism, and the binding mechanisms of Freundlich and Langmuir are listed as Equation (4) and Equation (5), respectively.

$$C_b = \alpha C_f \quad (3)$$

$$C_b = \alpha C_f^\beta \quad (4)$$

$$C_b = \frac{\alpha C_f}{1 + \beta C_f} \quad (5)$$

Where C_b and C_f are the concentration of chloride binding and free chloride, respectively; and α and β are the fitted coefficients.

EXPERIMENT PROCESS

Grade 42.5 Portland cement, polycarboxylic type high-performance water reducer, class I fly ash, grade S95 slag, water, sand, and limestone with the size of 5–20

mm were used as the main raw materials to produce concrete, and Table 1 shows the mix of the concrete. The sizes of the samples were 150 mm×150 mm×150 mm and 150 mm×150 mm×400 mm. Only one side of the concrete sample was left for testing; the other sides were sealed with epoxy. There were three stages for a test cycle, and the cycling time was about 72 hours. The spray stage was set for 50 minutes, and the last period was for a drying test. Temperature and duration time were set as 40°C/30 h, 50°C/12 h, and 60°C/30 h. The salt solution concentration was 5%, and the wind speed was set as 3 m/s. The size of the sample for establishing the correlation between free chloride and chloride binding was no more than 2mm, and the mass of the samples put into a conical flask was about 10 grams. The solution concentrations with a volume of 100 milliliters were set as 0.5%, 1%, 2%, 3%, 5%, and 10%. The samples were immersed in solutions for six months, and the solution concentrations were measured after the test. A profile grinding machine (model PF1100 produced in Denmark) was used to prepare the powder, and its fineness passed the sieve with the size of 75 μm. The chloride content in concrete was tested according to China standard JTJ 270-1998. The profile grinding machine and samples were as shown in Figure 1.

Table 1. Mix Proportion and Compressive Strength of Concrete / (kg·m⁻³)

Con- crete	Cement	Slag	Fly ash	Sand	Lime- stone	Water	Water reducer	Compressive strength for 28/60 d /MPa
C20	220	65	60	780	1030	176	3.9	25/29
C30	290	50	60	730	1050	164	4.2	33/36
C40	355	70	45	710	1060	155	4.5	45/49
C50	375	85	35	720	1085	152	5	53/58

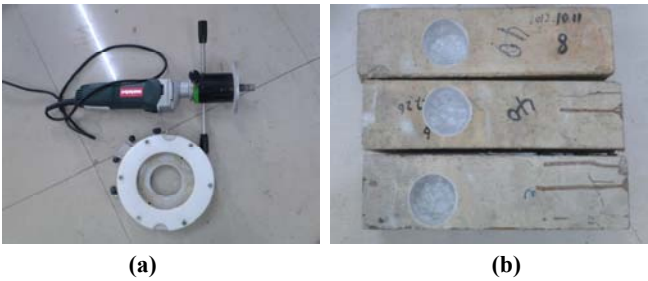


Figure 1. Profile grinding machine and sample.

RESULTS AND DISCUSSION

Time-varying law of chloride concentration on concrete surfaces. To discuss the rationality of the time-varying law of the chloride concentration model in an artificial simulation environment and a natural environment, this study took C20

and C50 concrete as examples to research with enough depth. The chloride concentration in concrete with time was discussed, as shown in Figure 2.

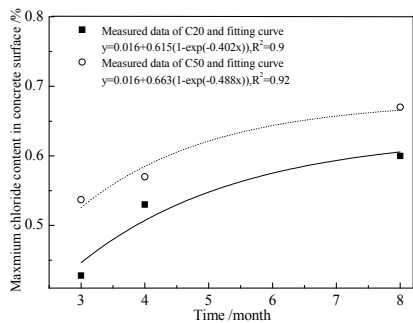


Figure 2. Maximum content of chloride in concrete surface with time.

From Figure 2, it can be seen that the maximum content of chloride of concrete surfaces increased with time, and finally tended to be a stable value. The higher the concrete was, the more maximum content of the chloride of the concrete surface was. With an increase of time, the final value of the chloride maximum content tended to be the same value. The fitting curve perfectly accorded with the measured data, which indicated that the modified exponential function model could be used to describe the time-varying law of the chloride content of the concrete surface. In fact, the change of the chloride content was an accumulating process with time, and the value finally reached a quasi-equilibrium state. When the concrete was wet, the chloride salt solution was absorbed by capillary suction of the unsaturated concrete, and the chloride ion was absorbed into the concrete and supplemented the consumption. With the drying process, the water transferred from the concrete in the form of vapor under the condition of the pore water without sustaining continuous status, and the chloride salt was concentrated and crystallized in the concrete. Concentration of the chloride at a certain depth in the concrete reached a new equilibrium state with the drying-wetting process, and the corresponding convective depth and limited chloride concentration were generated, which was related to the drying-wetting time ratio. The maximum chloride concentration in different grades of concrete was different, which may be caused by the difference of the micro structure and pores in the concrete. Generally speaking, the micro pore ratio of the total porosity in concrete increased with the strength of the concrete, so the stronger the capillary action was, and more chloride ions were adsorbed into the concrete in the initial period. However, the lower strength grade of concrete had more porosity, which gave it a stronger ability to adsorb more chloride ions.

Correlation between the free chloride and the chloride binding. Since the chloride salt concentration of the solutions under artificial simulation and natural environments were different, it is necessary to discuss its influence on the chloride

concentration of the concrete surface. To set optimal chloride concentration of the solution, the correlation between free chloride and chloride binding was investigated. Fitting curves of the Langmuir and Freundlich adsorption mechanisms were used to discuss the adsorption of the chloride in different concrete and were fitted to the measured data, as shown in Figure 3 and Figure 4.

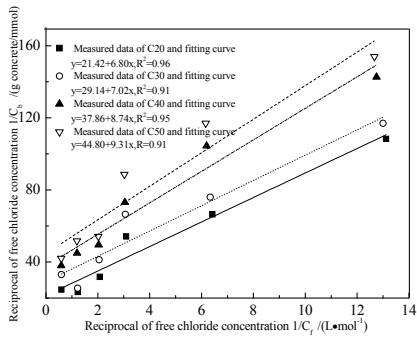


Figure 3. Fitting curve of Langmuir isothermal adsorption of chloride ion in concrete.

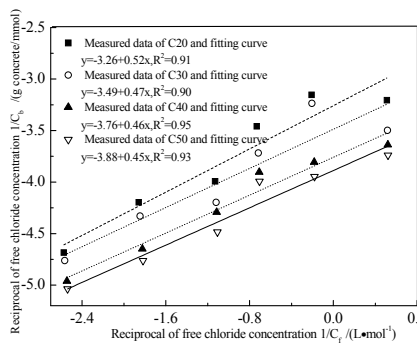


Figure 4. Fitting curve of Freundlich isothermal adsorption of chloride ion in concrete.

Figure 3 and Figure 4 reveal that the correlation between free chloride and chloride binding is preferable. Although both adsorption mechanisms could be used to describe the change in chloride adsorption, the linear function is not suitable to describe the correlation between free chloride and chloride binding. The chloride binding content increased with the increase of total chloride content, which may be caused by more chloride ions being adsorbed by hydration of the cement. Moreover, the content of the chloride binding decreased with the increase in concrete strength, which may be due to two aspects of the coupling effect. On the one hand, the higher-strength grade concrete contained more cementitious material, so there was more hydration that could adsorb more chloride ions. On the other hand, more hydration improved the density observably and could reduce

the pore channels for the chloride, so there were few chloride ions accessing into the concrete.

CONCLUSIONS

From the above discussion, some conclusions can be made:

(1) The maximum chloride content of the concrete surface increased with time, and finally tended to be a stable value. Compared with low-strength concrete, the higher-strength concrete contained more chloride in the initial period.

(2) A modified exponential function could be used to describe the time-varying of the chloride content of the concrete surface, and the fitting curves perfectly accorded with the measured data.

(3) Correlation between free chloride and chloride binding was preferable, and the adsorption mechanisms of Langmuir and Freundlich could be used to describe the correlation. The content of the chloride binding increased with the total quantity of chloride and reduced with increase in concrete strength.

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Chlorine Ion Content Distribution of Concrete Surfaces in a Natural Environment

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ABSTRACT

Based on Fick's Law of Diffusion, this research created a correlation model between the chlorine ion content in concrete surfaces and the elevation and distance away from the ocean in a natural environment. The real-time monitoring data of concrete chloride erosion in a natural environment were used to validate the rationality of the proposed correlation model. In addition, the variations of the chloride diffusion coefficient in concrete and the maximum chlorine ion content in a concrete surface along an elevation were investigated. The results showed that the chlorine ion content in concrete in a natural environment significantly changed and correlated with the elevation and distance from the ocean. The variation of chloride ion content in concrete surfaces along an elevation can be characterized by an S curve, and the variation of chloride diffusion coefficient of concrete along an elevation had a Gaussian distribution.

Key Words: Concrete; natural environment; elevation; content

INTRODUCTION

Chloride erosion affects the durability and service life of reinforced concrete. A great deal of research has been performed on this topic (Tamimi 2008; Song 2009; and Shi 2012). In et al. (Jin 2008) investigated the permeation mechanism and computational model of chlorine ion in concrete in an unsaturated condition. American NT Build 443 Standard provided a chloride erosion model on the basis of Fick's Second Law,

as shown in Equation (1) (NT Build 443 1995). According to original condition and boundary conditions, the analytical solution of Fick's Second Law can be expressed in Equation (2).

$$\frac{\partial C}{\partial t} = \frac{\partial}{\partial x} \left(D \frac{\partial C}{\partial x} \right) \quad (1)$$

$$C(x, t) = C_0 + (C_s - C_0) \left[1 - \operatorname{erf} \frac{x}{2\sqrt{Dt}} \right] \quad (2)$$

Where, $C(x, t)$ is the concentration of chloride at t time and x distance away from concrete surface; C_s is the chloride concentration of the concrete surface; C_0 is the original chloride concentration within concrete; D is the chloride diffusion coefficient in concrete; t is the time of structural exposure/testing; x is the depth away from surface; $\operatorname{erf}(u) = \frac{2}{\sqrt{\pi}} \int_0^u \exp(-\lambda^2) d\lambda$ is the error function, and the boundary conditions

are $C_{(0,t)} = C_s$, $C_{(\infty,t)} = C_0$, $C_{(x,0)} = C_0$.

By analyzing the above model, it can be concluded that the chlorine ion content in concrete is an important parameter for research into minimizing concrete erosion. In fact, the erosion of reinforced concrete structures by ocean chloride environments correlates with the vertical height over the ocean surface and the distance away from coast. Ann et al. (2009) concluded that the chloride ion content in concrete surfaces significantly affected the erosion of steel bars in concrete structures and proposed a simplified time-varying model of chloride ion content in concrete surfaces. Zhang et al. (1998) studied the chloride migration regularity in the inner of surface-treated concrete using a finite difference method and compared their results with Fick's Diffusion Law. Song et al. (2008) investigated the impact of concrete constitution, curing mode, environmental conditions, and exposure time on chloride ion concentration of the concrete surface. By comparing the existing outcomes of the chloride diffusion coefficient and the chloride concentration of concrete surfaces, they proposed a more practical logarithm with a time-varying model for chloride concentration of concrete surfaces.

Andrade et al. (1997) proposed a depth model in the concrete surface layer through analyzing the surface layer. Their research results revealed that the chloride diffusion coefficients of the concrete surface and in the concrete were different. The diffusion coefficient ratio of the concrete surface to the concrete interior characterizes the differences in surface property. Andrade et al. (2013) used resistivity to describe the anti-chloride erosion of concrete. Their results indicated that resistivity describes the chloride transport within concrete and could be correlated with time in the form of time square root. When the distance from the coast exceeds a limit, the chloride content in the atmosphere is mainly correlated with its distance away from the coast, and the impact of height in the vertical direction is negligible. If this paper assumes that chloride in the surface of the concrete is away from the coast and is mainly

influenced by the chlorine ion content in air, then it should be a function of distance, as expressed in Equation (3).

$$C_s = f(l) \quad (3)$$

Accordingly, concrete structures near the coast or within the ocean can be divided into an atmosphere zone, an underwater zone, a splash zone, and a tidal zone along the vertical direction based on their location environment. The discrepancy of different zones features the distribution of concrete surface chloride concentration with height. Chloride erosion of underwater concrete mainly depends on diffusion. Thus, the chloride concentration of concrete surfaces can be regarded as a function of seawater chloride concentration. The splash zone and tidal zone mainly depend on surface absorption and deep diffusion, and their characteristics are mainly presented in concrete surface chloride concentration and saturation of concrete, the ratio of seawater saturation, the chlorine ion content of the surrounding environment, and other related factors (Hu 2010).

Yao (2007) suggests that existing concrete saturation and dry-wetting time proportion are interrelated, and a certain dry-wetting time proportion corresponds to a certain concrete saturation. Dry-wetting time proportion is also related to periodic changing tides, which must be a function of height in a featured zone of concrete structure along the vertical direction. Therefore, this study concluded that the effects of concrete saturation, dry-wetting time proportion of seawater, and environmental chlorine ion content on chloride concentration of concrete surface are a function of vertical height h , as shown in Equation (4). In addition, since the effect of the atmosphere zone is mainly through absorbing chloride in the air, the chloride concentration of the concrete surface should be related to its vertical distance from the sea surface and can be expressed with Equation (3).

$$C_s = g(h) \quad (4)$$

Equations (3) and (4) relate concrete chloride concentration and vertical distance from the sea surface as well as the distance from the coast. The details refer to the simulation curves below.

Based on Fick's Diffusion Law and using the real-time monitoring data of chloride erosion within concrete structures under a natural chloride environment, this paper correlates concrete chlorine ion content with elevation and distance from the coast. In addition, the variation rules of the chloride diffusion coefficient and the maximum surface chlorine ion content along an elevation were also investigated.

EXPERIMENTAL PROCESS

A core machine was used to sample a field concrete structure. Then, a PF1100 profile grinder made in Denmark with 1mm resolution collected samples that were passed

through a $75\mu\text{m}$ square-mesh sieve. Concrete chloride concentration and water-soluble chloride were tested according to Concrete Test Specification of Port and Waterway Engineering (JTJ 270-1998). Figure 1 shows the profile grinder machine and the sampling process.



Figure 1. Profile grinder and field samples.

ANALYSIS AND DISCUSSION

Variation of chlorine ion content in concrete surface versus elevation. To validate the hypothesis that chloride ion content of a concrete surface varies with the change in vertical height, this research tested concrete chloride concentration along the vertical direction. Figure 2 shows the variation of chlorine ion content in concrete as a function of height. Figure 3 shows the variation in the concrete chloride concentration and diffusion coefficient with height. Figure 4 graphs wetting time as a function of height.

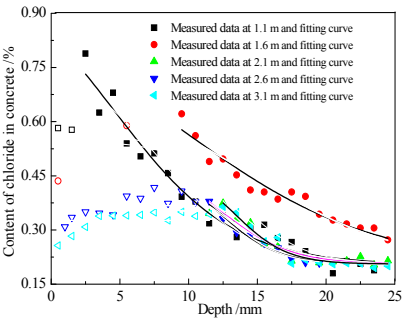


Figure 2. Chlorine ion content in concrete versus elevation.

From Figure 2, we see that the chlorine ion content of field harbor concrete significantly varied with height. When the concrete was in a tidal or shallow zone near sea level (1.1m), the chlorine ion content within 2.5mm of the concrete surface was generally stable and reached the maximum value at 2.5mm. This can be expressed in terms of Fick's Diffusion Law. When the concrete structure was located

at a tidal zone or splash zone (1.6m), the distribution of concrete chlorine ion content was similar to above except that the chloride convection zone of the concrete surface and the concrete chlorine ion content increased, which could be explained that during the wetting process of the above concrete, sea water saturated the interior due to capillary action. At the original drying period of ebb tide, the chloride solution in concrete can be transported toward the surface in liquid form. The filled pore water can then evaporate and increase the chlorine ion content due to concentration, hysteresis, and crystallization effects. This leads to a bilateral diffusion of chloride in concrete.

When concrete is re-wetted, seawater will quickly enter concrete with capillary action and, therefore, supplements the chlorine ion content at the concrete surface. At the beginning of wetting, the relative pore solution content of surface chlorides may be higher than that of the external solution due to evaporation and may diffuse towards the surface. But because convection is dominant, the recharge rate of the surface chloride is higher than its diffusion rate towards the surface and interior, which results in more chloride entering the concrete. Repeated dry-wet cycles increase the chloride concentration above semi-balance. The convection depth and content of chloride increase with dry-wetting time.

Due to the blocking effects of the concrete pore and the hysteresis effects of chloride diffusion, the above situations occur only at the concrete surface. However, the deep pore saturation changes little. Therefore, Fick’s Diffusion Law still can be used to characterize its regularity. When concrete is located in the atmosphere zone (2.1m, 2.6m, and 3.1m), the chloride ion content at a larger zone of concrete surface is generally constant, but the chloride diffusion within a deeper location applies to Fick’s Diffusion Law because concrete chloride in the atmosphere zone mainly comes from lower chloride resources such ocean fog, rain, and atmosphere. Therefore, the corresponding concentration gradient is also lower due to smaller diffusion driving forces, which can be identified from the wetting time ratio of concrete’s structural environment, as shown in Figure 3.

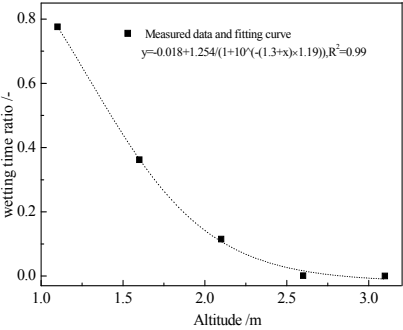


Figure 3. Curve of wetting time ratio with altitude.

As shown in Figure 3, the annual wetting time ratio of concrete structure decreased with increasing height over sea level and eventually approached zero at a certain elevation. The variation curve of wetting time ratio versus height indirectly validated the rationality of the above inference. From the variation tendency of wetting time ratio in Figure 3, theoretical computational values have a fairly good fit with a dose response function curve based on Sigmoidal fit, which indicated that the wetting time ratio in a certain elevation could be characterized by this function and, accordingly, provides an approach to obtain the tidal wetting time ratios of concrete structures at various elevations.

To better investigate the impact of elevation on concrete chloride, this research studied the variation of concrete surface chlorine ion content and diffusion coefficient as a function of elevation. The results are shown in Figure 4.

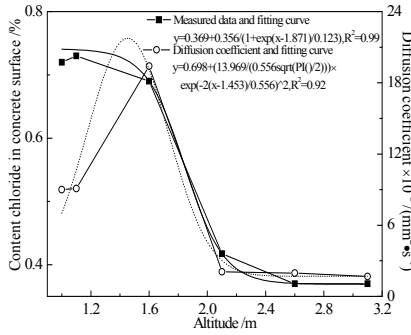


Figure 4. Curve of chlorine ion content and diffusion coefficient in concrete surface with altitude.

As shown in Figure 4, both the concrete surface chlorine ion content and the diffusion coefficient significantly changed with increases in elevation, but there was a slight difference between them. At the beginning, the chlorine ion content of the concrete surface slightly increased with an increase in elevation and maintained a certain value for a while, then quickly decreased. The overall variation regularity could be characterized by an S curve, as expressed in Equation (5). Meanwhile, the concrete chloride diffusion coefficient versus elevation increased and then decreased. The variation curve can be characterized by a normal distribution, as expressed in Equation (6).

$$y = a + b / (1 + \exp((x - x_0) / c)) \quad (5)$$

$$y = y_0 + \frac{a}{b\sqrt{0.5\pi}} \exp\left(-2\left(\frac{x - x_0}{b}\right)^2\right) \quad (6)$$

Variation of chlorine ion content in concrete surface versus horizontal distance away from ocean. We studied the concrete chlorine ion content as a function of horizontal distance from ocean, as shown in Figure 5. This value rapidly decreased with an increase in distance away from the coast and tended to stabilize after a certain distance. The overall change could also be described by the theoretical fitting curve in Figure 5 that shows the chlorine ion in the concrete surface was from continuous absorption and accumulation of chloride in air. Simply put, there is more chlorine in the air closer to the coast. Chloride content in air gradually decreases with distance. At a certain distance, chlorine ions in air are very limited, and the accumulation of chlorine ion in concrete surface is mainly provided by ocean fog and rain.

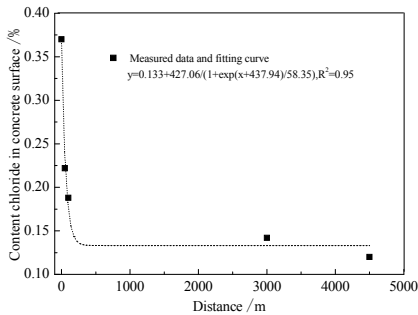


Figure 5. Curve of the maximum chlorine ion content in concrete surface with distance from coast.

CONCLUSIONS

- (1) The variation in chlorine ion content at a concrete surface changes with elevation and can be characterized by an S curve.
- (2) The variation of the diffusion coefficient of concrete chlorine ion changes with elevation and shows a Gaussian distribution.
- (3) The chlorine ion content in concrete significantly changes along the vertical direction. In a tidal zone or a splash zone, the chlorine ion content in concrete surfaces was generally constant. A maximum value of chlorine ion content exists, which is caused by the alternate drying and wetting processes in the above zones, as well as concentration, hysteresis, and crystallization effects of chloride solution in concrete.
- (4) The maximum content of chlorine ion at concrete surface rapidly decreases with distance from the coast and gradually stabilizes after a certain distance. This is because the chlorine ion in concrete surface results from continuous absorption and accumulation of chloride in air, and the chlorine ion content in air decreased with distance from coast.

ACKNOWLEDGMENTS

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Experimental Research on Impact and Flexural Behavior of RC Beams Strengthened with High Strength Steel Wire Mesh and High Performance Mortar

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ABSTRACT

The emerging strengthening technology of high strength steel wire mesh and high performance mortar is widely used in the field of seismic strengthening, bending and shear strengthening, and reinforcement of beam-column connections in China. In this paper, through the experimental research on impact and flexural behavior of RC beams strengthened with high strength steel wire mesh and high performance mortar, two different stress responses under different stress modes were evaluated. Drop hammer impact test was conducted on the B1 beam and static loading test was conducted on the B2 beam. Through the analysis of the failure mode, the strain of the steel and the mortar, and the change of deflection under different loading conditions, the difference was obtained under the two loading modes. The results showed that the beam under the impact resisted the instantaneous impact load, which reached the maximum quickly and then rapidly decreased to the minimum. The transfer of the impact energy was slightly slower than the change of the load. Additionally, the structure deformation of the beam under the static loading increased with the increasing load.

INTRODUCTION

High strength steel wire mesh and polymer mortar strengthening technology has been widely used in Korean structure and bridge engineering since the early of the century. Then the technology was introduced to China, domestic scholars carried out thorough research on material properties of high strength steel strand and polymer mortar, and the strengthening technology of reinforced concrete beam, plate, column, which plays a key role in the popularization and application in the reinforcement field in China. This paper focuses on comparing the performance of high-strength steel wire mesh and high-performance mortar strengthening reinforced concrete beam between impact and static loading.

TEST PROGRAM DESIGN

Test component

Two reinforced concrete beams are designed according to the concrete structure design code, whose number are B1, B2 with the size of 2800mm×200×400 (l×b×h, bearing distance: 2400mm). The strength grading of concrete of beam is C30, and the rebar grade is HRB335. After the completion of the reinforced concrete beams' maintenance, high-strength steel wire mesh and high-performance mortar

technology was used to reinforce the beams. The type of the high strength steel wire is 6×7-IWS. The diameter of steel wire is 3.2mm. The longitudinal spacing between the steel wires is 30mm, and the lateral spacing is 40mm. The grade of mortar with the incorporation of polypropylene fibers is M50, and the detailed design is shown in Figure1.

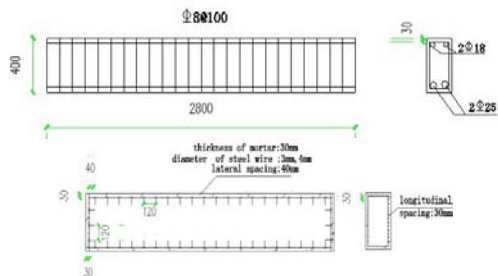


Figure 1. Design of the Components

Material properties of the test

Ductility and crack resistance of mortar is improved with the incorporation of polypropylene fibers. The detailed properties of the mortar were shown in Table 1.

Table 1. Properties of the mortar

Mortar strength grade	Consistency (mm)	Compressive strength of 7 days (MPa)	Compressive strength of 28 days (MPa)	Flexural strength of 7 days (MPa)	Flexural strength of 28 days (MPa)
M50	70~90	36	56.1	7.5	10.3

The material properties of concrete and reinforcing bar: the material properties including the tensile yield strength, ultimate strength of steel and the cube compressive strength, cylinder compressive strength of concrete and are given in Table 2.

Table 2. Properties of the concrete and steel (MPa)

The number of test specimen	B1	B2
The cube compressive strength f_{cu}	31.6	41.0
The cylinder compressive strength f_{ccu}	22.2	25.6
Φ25 The tensile yield strength of steel f_{sy}		398.3
Φ25 The ultimate strength of steel f_{su}		533.9
Φ18 The tensile yield strength of steel f_{sy}		438.9
Φ18 The ultimate strength of steel f_{su}		550.3
Φ8 The tensile yield strength of steel f_{sy}		337.2
Φ8 The ultimate strength of steel f_{su}		452.1
Mortar strength	49.1	52.2

Devices of test and measurement system

The beam was hinged at both ends and the bearing distance was 2400mm when the beam was installed on the test table. When the component was prefabricated, the steel bar strain gages should be embedded in both side of the longitudinal tensile and compressive on the triple points and mid-span of the beam, numbered 1 to 6. Three displacement sensors were set on the quartile and mid-span of the beam to determine the deflection change, numbered 1 to 3, as shown in Figure 2. Meanwhile, in order to measure the strain of the mortar during the process of the test, six strain gages were set on the surface mortar of the reinforcement layer. Four strain gages whose number were 1,3,4,6 were placed on the same position as the steel bar strain gage, the other two strain gages were set on one side of the triple points and mid-span numbered 2,5, the details of the arrangement were shown in Figure 2.

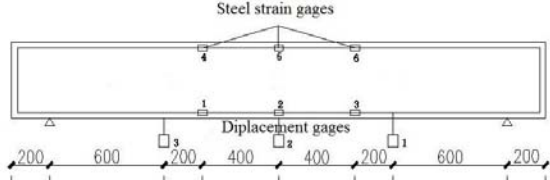


Figure 2. The arrangement of steel strain and displacement gages

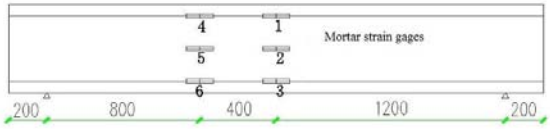


Figure 3. The arrangement of the mortar strain gages

Specific text program

The drop hammer test sketched on the B1 beam, which was carried out in school of civil engineering, Hunan University, as shown in Figure 4. The drop hammer was located on the mid-span of the beam, hammer diameter is 200mm, and the impact load was changed through adjusting the mass or height of the hammer. The weight of the impact hammer is 188kg, the weight of the mass is 195kg, the total weight is 383 kg, the impact height is 13 m, the speed is 15.96 m/s and the impact energy is 48794.2 J. In order to study the process of structural damage, the test was recorded by the high-speed camera synchronously.

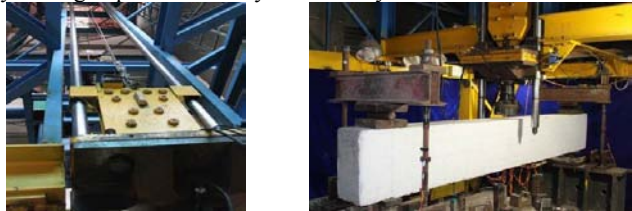


Figure 4. The device of impact test

The B2 beam was tested by the static bending test, using jack exert concentrated load on the mid-span of the beam. The experimental data was collected

by the strain gages installed in the beam and displacement device, as shown in Fig. 5.



Figure 5. The device of static test

THE EXPERIMENTAL PHENOMENA AND ANALYSIS

Analysis of the beam impact test

With the designed impact energy, the top impact area of the B-1 beam was crushed and parts of the mortar surface had peeling off under the impact load. As is shown in Figure 6(a), the characteristics of the impact load is huge and instant effect on the component, the contact zone played a buffer role dissipating parts of energy, then transfers energy to the whole structure generating integrated deformation. The distribution of cracks on the lateral sides of the beam is shown in Figure 6 (b), (c). There were large amounts of uneven distribution of micro-fractures in the impact area of the two oblique directions of 45 degrees. Three main cracks were generated on the bottom of the impact zone extended to the top of the beam with the maximum crack width of 9mm. The distribution of cracks on the bottom of the beam under the impact load is shown in Figure 6 (d). Multiple cracks has been extended through the bottom of the beam with the maximum crack width of 9mm meaning that the impact energy transferred to the bottom of beam and causing damage. Consider the overall failure state of the beam, the failure mode of the beam is mainly bending failure except the impact damage in the impact area under the impact load, which means the reinforcement layer has played a very good energy dissipation effect, changing the failure state of the original components and improve the impact resistance properties of the beams.

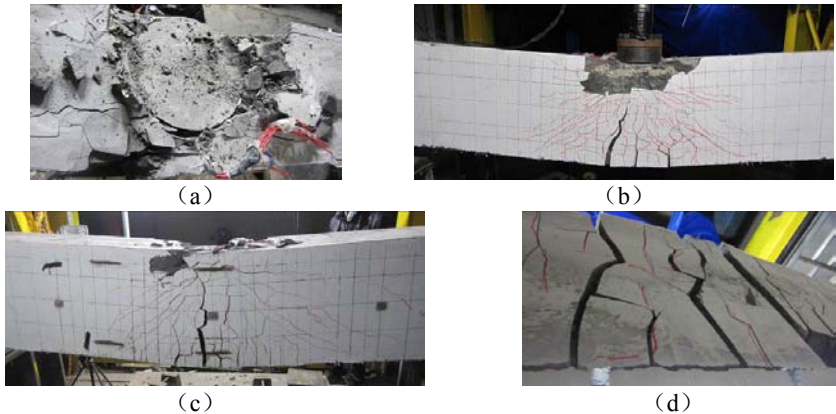


Figure 6. The failure state of B1 Beam

Analysis of the beam static test

The failure pattern of B-2 beam after the bending capacity test is shown in Figure 7. The area of the jack loaded on the top of the beam is shown in Figure 7 (a), because the static load was much smaller than the impact, and there were no load crushing destruction and instant impact effect. The distribution of cracks on the lateral sides of the beam is shown in Figure 7 (b), (c). Because there were large amounts of uneven distribution of micro-fractures on the surface of the polymer mortar, the load of the first bending cracks cannot be observed during the loading process. With the increase of load, a primary flexural crack appeared in the mid-span section, and the crack extended continuously along the cross section of altitude as its width was increased constantly. When the load reached 260.9kN, the longitudinal reinforcement across the tension control section went into the yield condition. When the load reached 283.3kN, the primary flexural crack had extended to the compressive zone, there were a few horizontal cracks appearing on the surface of the mortar's compressive zone showing the sign of crushing, at the same time slight tear sound can be heard at the bottom of the beam mid-span continuously. As the load reaching close to the limit load, the width of the crack and the mid-span deflection did not grow fast. When the load reached the ultimate load, the mortar on the compressive zone has been crushed a certain area, the deflection of the mid-span was about 18mm with the main bending crack width of about 1.8mm. Since then, continue to load, taking the mid-span deflection as the control factors, with the increase of deflection, load falling slowly, and the width of cracks increasing rapidly. When the deflection of the mid-span causing by the load increasing reached about 55 mm, then he load dropped to 260.7kN, which is only about 89% of the ultimate load, the ratio of the mid-span deflection and effective span (2400 mm) is about 1/43, which is more than 1/50. At this moment, compressive zone area appeared larger mortar crushed with the main bending crack width of 25mm, which meaning the specimen has been destroyed, and the failure pattern is bending damage, the failure state of B2 Beam is shown in Figure 7 (c).

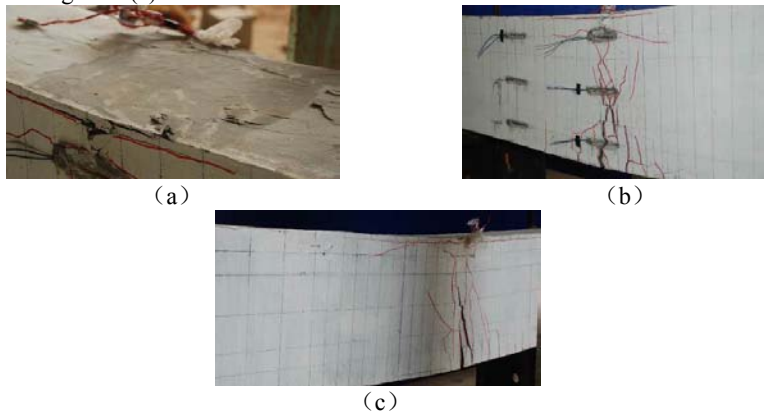
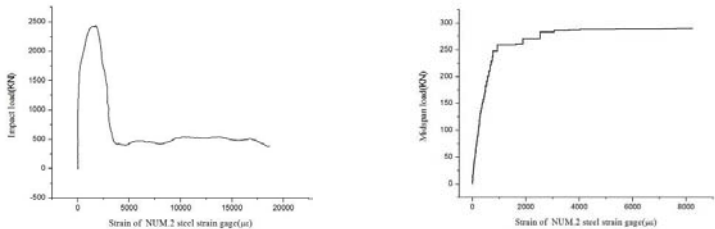


Figure 7. The failure state of B2 Beam

Analysis of steel strain

In order to analyze the strain of the beam under two different test modes

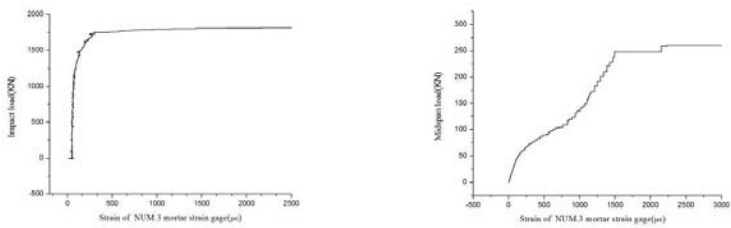
comparatively, the strain of steel strain gages numbered 2 on the mid-span of B1 beam and B2 beam were studied, as shown in Figure 8. The strain of No.2 strain gage of B1 is shown in Figure 8 (a), when the impact hammer touched the top of the beam, the impact load reached the maximize constantly, meanwhile, the steel strain was not the maximized, which demonstrated that reinforcement layer has dissipated large amount of impact energy, well protected the reinforced steel and lowered the speed of the original strain increase. After the peak load, the impact load decreased rapidly while the strain of the reinforced steel increased continuously, and steel yielded until it reached the ultimate strain, finally reinforced lost the ability to work. The strain situation of steel gages numbered 2 on the mid-span of B2 beam is shown in Figure 8 (b), due to the different load modes, the static load increased gradually, which was totally different from the instant effect caused by impact load. Before the load reached 250kN, the strain of the steel appeared to be a linear relationship which was increased with the load. When the load reached between 250kN and 270kN, the steel strain appeared obvious flow and the steel yielded at this moment, as the load increased, the steel strain reached ultimate strain, this is typical of a significant flow of steel bar stress - strain relationship. In summary, the overall energy consumption ability request of the structure withstand the impact load is much higher than the common structure. The reinforcement layer dissipates most of the impact energy and ensures the safety of the original structure.



(a)Strain of No.2 strain gage of B1 (b) Strain of No.2 strain gage of B2
Figure 8. The curve of steel strain

Analysis of surface mortar layer strain

From the mortar strain gages No.3 on the same location as the steel strain analyzing the differences between the two strain gages. The strain situation of mortar gages of B1 beam is shown in Figure 9 (a); the impact load reached the maximum instantly after suffering the impact load. The mortar surface is the first defense of structural safety, when load reached the maximum, the mortar strain rapidly increased to maximum until the construction crushed. The curve of mortar strain is shown in Figure 9 (b), when B2 beam suffered static load, which is different from the instant change of B1 beam, the mortar strain increased with the load increasing. When the strain reached peak strain, the mortar crushed gradually. The comparison above showed that the stress of the structure response is completely different under disparate load.

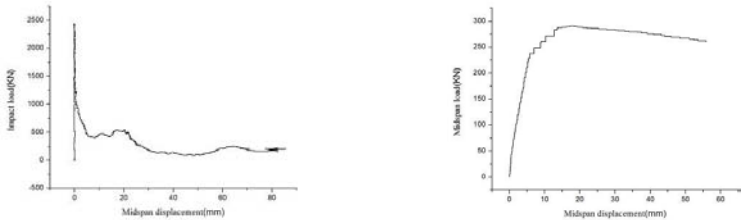


(a) Mortar strain of NUM.3 strain gage of B1 (b) Mortar strain of NUM.3 strain gage of B2

Figure 9. The curve of mortar strain

Analysis of the mid-span deflection

The curve of mid-span deflection recorded by the displacement meter during the test process is shown in Figure 10. When the beam suffering the impact load, the impact load reached the maximum constantly and then decreased rapidly, the huge impact energy began to transform from the top of the beam to the body of the beam in a very short time as from Figure 10(a). The deflection of the mid-span of the beam increased gradually as the load decreased, this was the process of the impact energy transforming, the impact energy dissipated slowly while it transformed from reinforcement layer to the original structure, which caused the overall deformation of the structure, and the deflection of the mid-span reached maximum. Comparing with the curve of mid-span deflection shown in Figure 10 (b), when the beam suffered static load, the deflection of the mid-span increased slowly as the load increased gradually. When the load reached a maximum, the deflection increased sharply and achieved the maximum value gradually. In summary, the process of the static load increasing was also the process of the energy transformation, the response of the structure changed with the load. But in the impact test, the huge impact load loaded on the structure instantly, the energy transformed as the load decreasing, the response of the structure was lag behind the load change, this process compared with the static load was extremely short.



(a) Mid-span displacement of B1 (b) Mid-span displacement of B2

Figure 10. The curve of mid-span deflection

CONCLUSIONS

Based on the analysis of the results combined with data processing in this paper, the conclusions were shown as follow:

- (1) The impact test is a process of instant loading. The drop hammer touched

the top impact area of the beam and the huge impact energy transformed to the beam structure in an extreme short time, which raised the crush of the structure. The top impact area of the beam was locally crushed. A large amount of uneven micro-fractures appeared in the impact area of the two oblique directions of 45 degrees and three main cracks were generated on the bottom of the impact zone propagated to the top of the beam. Due to the dissipation effect of the reinforcement layer, the failure mode of the beam is bending failure not the impact failure.

(2) The static bending test was different from the impact test. When the beam suffered the concentrated load, the structure deformation generated gradually as the load increases. The vertical cracks almost appeared in the tensile area. The two primary cracks were the largest, and the cracks became smaller when approached to the sides. This failure mode is a typical bending failure.

(3) It was found that the structural stress responses always appeared instantly with bigger deformation from the analysis of tensile area reinforcement strain when the structure suffered impact load. It is quite significant to find the suitable method to improve the structural impact resistance and the energy dissipation capacity, which could reduce loss or slow down the damage. The structure suffered static load is a typical example of the variation of the structure changing with the load.

(4) According to the analysis of surface mortar layer strain, when the impact load loaded on the beam, reinforcement layer mortar was the first line of defense, which dissipated large amount of impact energy. Then the tensile mortar crushed instantaneous, highlight the instantaneity and destructive properties of the impact. The mortar crush under static load is a gradual change process.

(5) According to the comparative analysis of the mid-span deflection, due to the instantaneity of the impact load, the transformation of the impact energy was lag behind the change of the load, and the change of the mid-span deflection was lag behind the load change. As the load decreased, the mid-span deflection increased gradually. Meanwhile, the structure deformation of the beam under the static loading increased with the increasing load.

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Seismic Design Guidelines of Retrofitted Deep Concrete Coupling Beams with Laterally Restrained Steel Plate

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ABSTRACT

Existing deep reinforced concrete (RC) coupling beams with low shear span ratios and conventionally reinforced shear stirrups tend to fail in a brittle manner with limited ductility and deformability under reversed cyclic loading. Retrofitted coupling beams with laterally restrained steel plates (LRSP) have been demonstrated experimentally and numerically in the previous studies to be a viable method for improving the structural performance of existing deep coupling beams. In this paper, an original seismic design procedure of LRSP coupling beams is proposed and it consist of five main parts, which are (1) estimation of the ultimate rotation of the original coupling beams (2) estimation of axial force, (3) rotation capacity of bolt group, (4) rotation capacity of steel plate, (5) rotation compatibility of retrofitted coupling beam.

INTRODUCTION

Reinforced concrete (RC) coupled shear walls and core walls are widely employed as a lateral load resisting system for high-rise buildings to resist earthquake and wind loads. To ensure the desired behaviour of coupled core walls, seismic resistant coupling beams should be sufficiently strong, deformable and have good energy dissipation ability (Paulay, 1971). Many studies revealed that many existing deep coupling beams had limited ductility and tend to brittle shear failure which may lead to the serious failure of the whole lateral resisting system. However, little research had been conducted aiming at improving the deformability and reducing the strength degrading of existing reinforced concrete coupling beams. Harries et al. (1996) studied a shear strengthening method for coupling beams with a span-to-depth ratio of 3.0 and they found that the composite method of bolting with epoxy bonding to attach the steel plates both in the span and at the ends performed better. Su and Zhu (2005) studied a shear strengthening method for RC coupling beams with a span-to-depth ratio of 2.5. They proposed to strengthen the coupling beams by bolting the steel plate to two

ends of wall panels without adhesive bonding. However, most of the previous studies focused on the coupling beams with span-to-depth ratios larger than 2.0. Su and Cheng (2011) experimentally studied the use of a laterally restrained steel plate (LRSP) to retrofit deep concrete coupling beams with a span-to-depth ratio of 1.1. Their study revealed that the LRSP improved the deformation and energy dissipation of the deep coupling beams. Extensive experimental and numerical studies (Cheng and Su 2011a; Cheng and Su 2011b) have been conducted to investigate the structural performances of LRSP coupling beams under cyclic loading. Based on the results and findings from these studies, a design guidelines of ultimate strength of retrofitted deep coupling beams has been proposed (Cheng and Su, 2012). In this paper, deformation design guidelines of the retrofitted deep coupling beams are proposed.

LRSP RETROFITTING METHOD

The previous experimental study on LRSP deep coupling beams demonstrated that local buckling of steel plates is hard to avoid. A plate buckling control device, which is composed of steel angles (Figure 1), was suggested to mount to the beam span to suppress plate buckling under shear loads. To avoid adding extra strength and stiffness to the composite coupling beam, the lateral stiffeners are connected to a steel plate by bolt connections with slotted holes, which allow the two lateral stiffeners to freely rotate and move in the longitudinal direction. The advantage of using a plate buckling restraining device instead of adding stiffeners to the steel plates to control plate buckling is that the stiffness of the coupling beams would not increase. Yielding of the plates can be allowed for energy dissipation purposes. Experimental studies have demonstrated that for the specimens with the added buckling control device, a continuous shear transfer medium across the joints that was provided by the steel plate can continue to take a larger share of the load in the post peak region and alleviate concrete crushing in the compression region. As a result, LRSP coupling beams failed in a more ductile fashion.

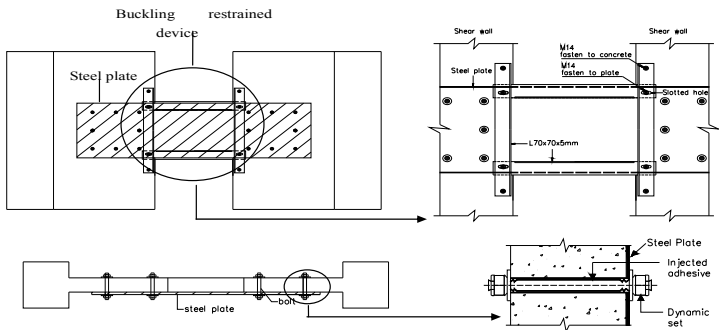


Figure 1. Retrofitting Scheme of LRSP Coupling Beam

PROPOSED DEFORMATION DESIGN GUIDELINENS

Application range of LRSP retrofitting method

Deep RC coupling beams may fail in shear or bending. Based on the flexural failure mode, the ultimate shear strength of the beam V_{u1} may be expressed as (Subedi, 1990):

$$V_{u1} = \frac{2h'_0}{l} A_s f_{ys} \quad (1)$$

The ultimate shear stress capacity is

$$v_{max} = v_c + v \cdot 0.79 \left(\frac{100A_s}{b h_0} \right)^{1/3} \left(\frac{400}{h_0} \right)^{1/4} \left(\frac{f_{cu}}{25} \right)^{1/3} \left(\frac{2h_0}{l} \right) + \frac{1}{r_{ms}} \frac{A_{sv}}{s_v} \cdot \frac{f_{yv}}{b} \quad (2)$$

The shear capacity of RC coupling beam can then be expressed as:

$$V_{u2} = v_{max} b h_0 \quad (3)$$

where A_s and f_{ys} are the area and the yield strength of longitudinal reinforcements respectively, A_{sv} , s_v , and f_{yv} are the area, the spacing and the yield strength of transverse stirrups respectively. f_{cu} is the compressive strength of concrete. b is the width, h'_0 is the distance between the top and bottom of the longitudinal reinforcements and h_0 is the effective depth of coupling beam. l is the span of the coupling beam. r_{ms} is the safety factor of reinforcement ($r_{ms}=1.15$).

Based on the experiments on deep RC coupling beams conducted by Paulay (1971), Subedi (1990) pointed out that when the shear strength V_{u2} is smaller than V_{u1} , brittle shear or diagonal splitting failure tends to occur. Fang (1988) carried out many tests on deep RC coupling beams and obtained the same conclusion with Subedi (1990). A ratio β is defined in Equation (4). when $\beta \geq 1$, deep coupling beam is likely to fail in brittle shear mode

$$\beta = V_{u1}/V_{u2} \quad (4)$$

The ratio of shear stress v_{max} to the compression stress f_{cu} is defined as:

$$\zeta = v_{max}/f_{cu} \quad (5)$$

Fang (1988) pointed out that ζ also affects the ductility of coupling beams. When ζ is higher than 0.15, brittle shear or diagonal splitting failure tends to occur. Figure2 shows the application range of the LRSP retrofitting method. In this range, β is between 1.0 and 2.0 and ζ is from 0.075 to 0.15. Deep RC coupling beams in this range tend to fail in a brittle shear mode and their ductility and deformability may need to be improved by LRSP retrofitting method.

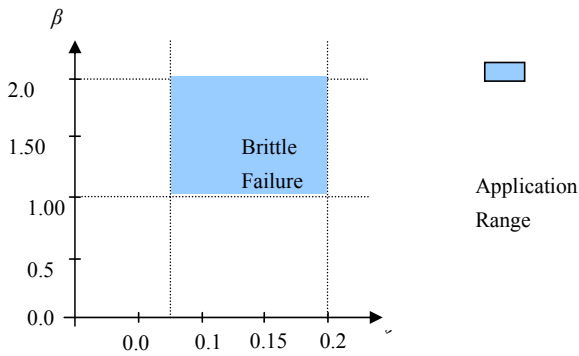


Figure 2. Application Range of the LRSP Retrofitting Method

Deep coupling beams with span-to-depth ratio of $1 \leq l/h \leq 2.5$ were designed as brittle shear failure mode and the ultimate strengths and ultimate rotation of the designed RC coupling beams were investigated. Full-scale NLFEA models were constructed with the same span ($l=1000\text{mm}$) and the same size of wall panels. The dimensions of the designed beams were set within a normal range to simulate the real coupling beams. The NLFEA models were divided into four groups, i.e. Group DB1.0, Group DB1.5, Group DB2.0 and Group DB2.5 with the span-to-depth ratios (l/h) of 1.0, 1.5, 2.0 and 2.5 respectively. The depth-to-width ratios (h/b) adopted were 2.0 and 4.0. It is noted that for a coupling beam, if β and ζ have been specified, the details of flexural reinforcements and shear reinforcements can be determined. When $\beta \geq 1$, the failure tends to be a brittle failure mode. In each design group, $\beta = 1.0, 1.5$ and 2.0 and $\zeta = 0.075, 0.1, 0.15$ and 0.2 .

Based on the numerical results, the rotations at peak load of each designed coupling beam were obtained. Figure 3 shows the variation of the rotation at peak load with the span-to-depth ratio. It can be found that the rotation at peak load is mainly affected by β and l/h . With the increase of l/h , the rotation at peak load is increased. With the increase of β , the rotation at peak load is decreased. If the details of reinforcements and material properties of the coupling beam have been given, the rotation of the coupling beam under certain loading can be obtained.

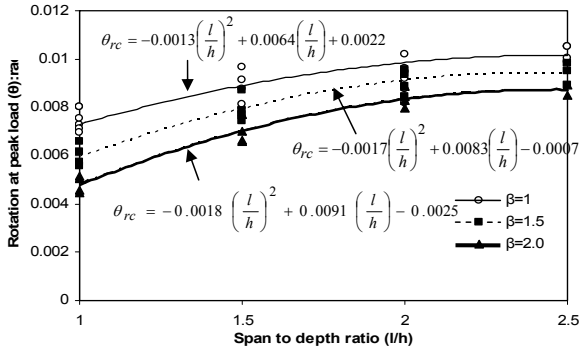


Figure 3. Estimation of Rotation at Peak Load with Span-To-Depth Ratio

Estimation of Axial Force

It should be noted that due to the differential longitudinal deformations between the steel plate and the RC beam, axial compressive force and tensile force are developed in the concrete and steel plate respectively. Paulay(1971) studied the diagonal tension failure mechanism of deep RC coupling beams, and he proposed the elongation of RC coupling beam due to the extension of reinforcements can be calculated by Eq (6):

$$\Delta_{lrc} = \frac{l}{A_s E_s} \int_0^l T(x) dx = \frac{V_{rc} l^3}{2 A_s E_s z} \left(1 - \frac{V_s}{3 V_{rc}}\right) \quad (6)$$

where V_{rc} and V_s are the shear forces taken by the RC coupling beam and transverse stirrups, respectively, and A_s and E_s are the steel area and elastic modulus of the longitudinal steel, respectively. Considering the horizontal deformation compatibility between the steel plates, bolts and RC beams, the total beam elongation Δ_{lrc} is equal to the sum of shortening of RC beams, extension of steel plate and the horizontal slip of bolt group. By invoking the horizontal force equilibrium, the axial force N_p induced in the steel plate and concrete can be estimated:

$$N_p = \Delta_{lrc} \frac{K_{rc} K_b K_p}{K_p K_b + 2 K_p K_{rc} + K_{rc} K_b} \quad (7)$$

$$K_{rc} = \frac{E_c A_c}{l}, \quad K_p = \frac{E_p A_p}{l}, \quad K_b = n_b K_{bl}$$

where K_{rc} , K_p and K_b are, respectively, the axial stiffness of the RC beam and steel plates and the horizontal stiffness of the bolt group at one end of the beam. A_c and E_c are the area and elastic modulus of concrete beam. A_p and E_p are the area and elastic modulus of steel plate. n_b is the number of the anchor bolts at the one end

of the beam and K_{bl} is the stiffness of a single bolt connection.

Rotation Capacity of Bolt Group

Figure 4 (a) shows a general arrangement of a group of bolts subjected to a general external load (F_x, F_y, F_θ) . The applied forces at the stiffness centre of the bolt group can be related to the applied forces at the beam-wall joint by Eq.(8):

$$(F_x, F_y, F_\theta) = (N_p, V_p, M_p + V_p(l_{bg} - l/2)) \quad (8)$$

where l_{bg} is the distance from the centre of the bolt group to mid span of the beam, N_p , V_p and M_p is the axial force, shear force and moment induced in the steel plate respectively. Given that the lateral stiffness of the i th bolt in the x and y directions are k_{xi} and k_{yi} , respectively, and the lateral movements of the bolt group at the stiffness centre are denoted by $(\Delta s_l, \Delta s_v, \theta_{sr})$, Su and Siu(2007) determined the slips of the bolt group based on the classical elastic theory as:

$$\begin{Bmatrix} \Delta s_l \\ \Delta s_v \\ \theta_{sr} \end{Bmatrix} = \frac{1}{-R_x^2 K_y - R_y^2 K_x + I_{zz} K_x K_y} \begin{bmatrix} I_{zz} K_y - R_y^2 & -R_x R_y & R_x K_y \\ -R_x R_y & I_{zz} K_x - R_x^2 & -R_y K_x \\ R_x K_y & -R_y K_x & K_x K_y \end{bmatrix} \begin{Bmatrix} F_x \\ F_y \\ F_\theta \end{Bmatrix} \quad (9)$$

$$K_x = \sum_i k_{xi}, \quad K_y = \sum_i k_{yi}, \quad I_{zz} = \sum_i (k_{xi} y_i^2 + k_{yi} x_i^2)$$

$$R_x = \sum_i k_{xi} y_i, \quad R_y = \sum_i k_{yi} x_i$$

Rotation Capacity of Steel Plate

Based on Timoshenko's beam theory, the plate rotation due to combined bending and shear can be estimated by Eq. (10):

$$\theta_p = \frac{V_p I_{bg}^2 \left[1 + 0.6(1 + \nu) \left(\frac{h_p}{l_{bg}} \right)^2 \right]}{3E_p I_p} \quad (10)$$

where ν , E_p , I_p and h_p are, respectively, the Poisson's ratio, elastic modulus, second moment of area and height of the steel plates and V_p is the shear force in the steel plate.

Rotation Compatibility of Retrofitted Coupling Beam

Slips of mechanical connectors in steel plate RC structures can significantly affect the load-carrying capacity of the structures. The difference in deformations between the steel plate and the RC counterpart is known as partial interaction. Figure 4 (b&c) illustrates the deformation relationships between the RC beam, steel plate and slippage of bolts. By comparing the initial and displaced locations of the stiffness centre of the bolt group, the slips of anchor bolts can be found. In

the subsequent discussion, the slips in the longitudinal, transverse and rotation directions are denoted respectively as Δsl , Δsv , and θsr . By considering the rotation compatibility between the steel plate and the concrete beam, the total chord rotation θ of the LRSP beam can be expressed as Eq. (11):

$$\theta = \theta_s + \theta_{sv} + \theta_p + \theta_{sr} \quad (11)$$

where θ_s is the rotation due to the change in the reference position from the beam-wall joint to the stiffness centre of the bolt group, θ_{sv} is the rotation caused by the transverse slip of bolts, and θ_p is the combined flexural and shear rotation of the steel plate. Because the longitudinal slip gives only a very small contribution to the rotation compatibility, its effect is neglected in the formulation of Eq.(11). According to Figure 4(c), the rotations θ_s and θ_{sv} can be derived as follows:

$$\theta_s = \theta_{rc} \left(1 - \frac{l}{2l_{bg}} \right) \quad (12)$$

$$\theta_{sv} = \Delta_{sv} / l_{bg} \quad (13)$$

where l_{bg} is the distance from the centre of the bolt group to mid span of the beam, and θ_{rc} is the chord rotation of the RC beam.

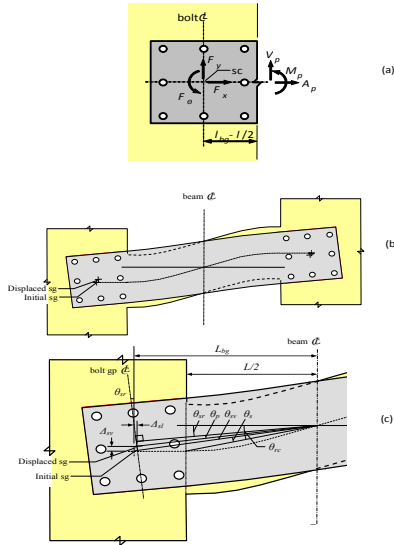


Figure 4. Longitudinal, transverse and rotation slips (a) bolt group and applied forces (b) overall deformations and (c) slip components

CONCLUSIONS

From the aforementioned experimental and numerical studies, it has been found that the anchors at the ends of the steel plates should be designed to be strong and stiff enough to transfer all forces from the steel plates to the wall regions. To ensure sufficient stiffness of the bolt group, the total rotation estimated by Eq.(11) is recommended to be not more than $(1.2\sim 1.5)\times \theta_{rc}$.

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Experimental Study on High-Early-Strength Precast Concrete

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ABSTRACT

High-early-strength precast concrete technology can improve production efficiency and save energy. The use of early-strength type polycarboxylate water reducers can shorten the setting time of concrete and steam curing time of precast concrete and can significantly improve the early strength of concrete. Early-strength superplasticizer BTC300 bearing with sulfonic acid groups and amide groups is synthesized at room temperature and has some significant functions, such as slow release, early strength improvement, and high water reduction. The tests of preparing C30 ~ C80 precast concrete show that compared with concrete with the normal superplasticizer BTC100, concrete with early-strength type superplasticizer BTC300 have higher 1–3 day average strength, whether under steam curing conditions or at room temperature, and the feature becomes more prominent as the strength grade increases. Also, the 28-day strength of concrete with BTC300 is slightly higher. GPC analysis shows that the active ingredients of BTC300 superplasticizer molecules are much higher and the content of small monomer polymers is low, so it has little influence on delay of cement hydration reaction. Hydration heat tests show that the second hydration exothermic peak with BTC300 is nearly 10 hours earlier than that of with BTC100; these proved that the function of BTC300 is relatively early strengthening.

INTRODUCTION

Although widely-used traditional heat-curing technologies can enhance the productivity of precast concrete, energy consumption is too large. In addition, it is possible to obtain high early strength and high strength by means of steam curing, autoclave curing, and infrared and microwave maintenance, but the structural defects of concrete materials increase, and it also has a certain impact on the durability of concrete products (Zhu 2012). Superfine cement with early high strength is used to prepare super early high-strength concrete, that is to say, 1-day strength concrete reached about 50% of design strength level and 3-day strength reached 90–100% of concrete design strength level. This makes the maturity of concrete strength shorten from 28 days to 3 days, which can increase work progress and greatly reduce the number of templates. However, the late strength of concrete can hardly continue to

grow, and it can even cause inverted shrinkage; also, the cracking sensitivity of concrete material will increase. Due to the significant energy requirements of superfine cement production, the costs become higher, so precast concrete is not really sustainable high-performance concrete. With the development of industrialization in architecture, concrete production in the 21st century continuously advocates resource-saving and environmental friendliness. Because precast concrete in construction is more and more widely used, the technology of improving the early strength of concrete has become increasingly important for ensuring the final performance of concrete. Further study on the technology of early strength of precast concrete is required. The quality and appearance of elevated and large-span prestressed bridges requires new technology of early strength and high strength precast concrete, the use of early strength polycarboxylate water reducers with super water-reducing rate, and the application of mineral admixtures and ordinary Portland cement. By adjusting the concrete mix, ultra-low water-cement ratio concrete can be achieved, the early strength of concrete can be much higher, and the late strength can grow steadily. Super early-strength concrete has good performance, which ensures that the concrete box girders, etc., do not have a honeycomb and pock phenomenon. For setting time to meet construction schedule requirements, rapid early strength development and also the setting time to reach the requirements of construction progress are important. Early strength increases quickly and later strength increases stably, which the precast concrete industry is very concerned about (Dvorkina et al. 2012, Cerulli 2006, Guo and Zhang 2012, Ma et al. 2009).

Foreign company Sika has specifically studied precast concrete superplasticizer and its early strength composite products. Polycarboxylate water reducer ViscoCrete 20HE was designed specifically for precast concrete synthetic (Wang et al. 2009). The use of early strength polycarboxylate water reducers can improve the durability of concrete and shorten the setting time of concrete; it can also significantly improve the early strength of concrete and shorten the steam curing time of precast concrete, etc. However, domestic study on early strength polycarboxylate water reducers has not reached ideal results; the methods of compounding of ordinary polycarboxylate water reducers and triethanolamine have only a limited effect (Du et al. 2011). In this study, the theory is based on the more sufficient reaction of polycarboxylate water reducers under the conditions of catalyst at room temperature by using early strength superplasticizer BTC300 synthesized at normal temperature to prepare high-early-strength C30 ~ C80 concrete. More active ingredients exist, and the actual water reducing rate can reach more than 40%. The theoretical basis relies on absorbability provided by carboxylic groups at an optimum ratio and powerful dispersed stability formed by sulfonic group and ether side-chains molecular structure. A great water reduction rate can reduce concrete viscosity in a strong diffusion power, and the amino and alcohol amine groups have a strong function of adsorption on the surface of cement base mineral particles. When BTC300 is applied to high-strength grade concrete construction in winter, the early strength properties of 1-day and 3-day are outstanding, and with the improvements of concrete design strength, the effects would be even more remarkable. Therefore, it is beneficial to energy savings to use precast concrete, reducing costs and making the production of the green precast concrete.

MATERIALS AND METHODS

Materials for synthesizing early-strength type polycarboxylic water reducer BTC300. The macro monomers in synthesis raw materials are methyl acrylic base polyethylene glycol 2400 and isopentenyl polyethylene glycol 2400, both of which have purity above 90%; small monomers are polymerization grade acrylic, sodium methyl allyl sulfonate, acryl amide, etc. Initiator is 27% hydrogen peroxide solution, catalyst is ascorbic acid, chain transfer agent is β mercaptopropionic acid, pH regulator is triethanolamine, and water is deionized water.

Materials of concrete. Cement is Jinyu P.O42.5 ordinary Portland cement. Stones are 5mm–20mm continuous grading gravels whose mud content is less than 1% and needle and flake-shaped content is less than 5%. Sand is medium and coarse river sand at a fineness modulus of 2.7, with a mud content is less than 2%. Admixtures are II level fly ash and S95 slag powder. Admixture is homemade early-strength type polycarboxylic water reducer named BTC300 at 40% and the saled standard type polycarboxylic water reducer of 40%KL100 mother liquor.

Synthesis of early-strength type polycarboxylic water reducer. Put methyl acrylic base polyethylene glycol 2400, isopentenyl polyethylene glycol 2400, and water at mass ratio of 1:1:1 into the 3-litre glass reactor to heat and mix, than add a moderate hydrogen peroxide solution of 27%. Use polymerization grade acrylic, sodium methyl allyl sulfonate, acryl amide and water, etc., to prepare a small-monomer solution A of about 30%, and use moderate ascorbic acid, β mercaptopropionic acid, and water to prepare catalyst solution B of about 1%. When the solution in the reaction vessel is slowly heated up and stable at $63 \pm 2^\circ\text{C}$, drip solutions A and B uniformly at the same time. Drip solutions off at about 2–2.5 hours, then cool the mixed solution after 0.5–1.0-hours of aging. Use 30% triethanolamine solution to neutralize the mixed solution. By diluting the solution, early-strength type polycarboxylic water reducer BTC300 of 40% can be prepared.

Property tests of cement paste mixed with polycarboxylic water reducer. (1)

Fluidity and fluidity maintaining property test of cement paste: A fluidity test of cement paste refers to “The Qualitative Test Method of Concrete Admixture Uniform” (GB807 2000). Take W/C=0.29. According to Appendix A of “Technical Specification for Application of Concrete Admixture” (GB50119 2003) to test the adaptability of concrete admixture to cement. Ensure that the initial fluidity is over 200mm. Add 261g water into a proper water reducer at a concentration of 40%, cement 900g. Use a cement paste mixer (NJ-160A) to rapidly stir for 2 minutes and slowly stir for 2 minutes, then rapidly inject mixed cement paste into a truncatedconical round model (Φ above=36mm, Φ below=60mm, h =60mm). Use a scraper to slick the top, and raise the truncatedconical round model in the vertical direction. Open a stopwatch at the same time, and let the cement paste free-flow onto a glass plate, then measure the maximum diameter of the flowing cement paste at two perpendicular directions. Take the average value as the fluidity of the cement paste. Leave the rest of the cement paste in the mixing pot and cover it with a damp cloth. One or two hours after adding the

water, stir rapidly for 4 minutes, then test the fluidity of the cement paste.

(2) *Viscosity test of cement paste*: The test instrument is a direct-reading type NDJ-5S rotary viscometer. Rapidly inject the mixed paste into specimen cups until all rotors have been submerged. Fix the fastening in the specimen cup, and start rotating the viscometer button, than read the viscosity reading.

Property routine tests of concrete mixed with polycarboxylic water reducer. Routine tests on water reducing rates, air content, benchmark for setting time, and compressive strength of concrete at mix proportion of cement : sand : stone = 390 : 785 : 1060. Slumps of both reference and tested concrete are controlled in 200~220mm. The testing method refers to relevant provisions in “Concrete admixture” (GB8076-97).

Mix proportion and property tests of early strength high performance concrete of C30 ~ C80 for precast elements.

(1) The slump of high performance concrete precast is controlled in 180–200mm. Unilateral amounts of material of several tested concretes for C30 ~ C80 are shown in Table 1. Each test uses both ordinary polycarboxylate water reducers BTC100 and early strength polycarboxylate water reducers BTC300 to conduct comparison tests, then the average compressive strength of concrete for 1, 2, 3, and 28 days is tested under standard curing conditions;

(2) Because precast concrete is usually maintained under steam curing after it is shaped, C40 groups are taken in a 55–60° C steam curing condition for 8 hours. After 4 hours of static stopping, demold after natural cooling, and test for 12 hours the compressive strength and average compressive strength of 28 days after late standard curing.

(3) For C80 high strength concrete, test the average compressive strength of 1-day and 3-days natural curing and 28 days after late standard curing under winter conditions of 0–5° C and summer conditions of 30–35° C.

Table 1. Amount of High Performance Materials for Precast Concrete (kg)

Strength Grade	C	FA	K	S	G	W	AD	SP	WB
C30	210	90	70	820	1045	165	0.15	0.44	0.445
C40	274	84	63	718	1095	165	0.18	0.40	0.390
C50	300	80	95	687	1075	164	0.20	0.39	0.347
C80	350	60	150	625	1065	150	0.30	0.29	0.268

GPC analysis of polycarboxylate water reducers. The testing instrument is a Waters 1515 Isocratic HPLP Pump/Waters 2414 Refractive Index Detector. Chromatographic column is in series form of Ultrahydrgel™120 Ultrahydrgel™250 and Ultrahydrgel™500 Column. Mobile phase is 0.10 mol/L sodium nitrate aqueous solution. Flow rate is 0.5 mL/min, column temperature is 40°C, and sample volume is 20.000 uL. Reference material is polyethylene glycol 1000, 2000, 5000, 10000, 50000, 100000, etc.

Test of influence of polycarboxylate water reducer on cement hydration heat. According to “Cement Hydration Heat Measurement Method” (GB/T12959-1991), the instrument to test and analyze cement hydration heat was ToniCAL Model 7338, and the main test was to analyze the cement continuous hydration heat of blank and the ones added with BTC100 or BTC300.

RESULTS AND DISCUSSION

Performance of cement paste mixed with polycarboxylate water reducer. Figure 1 shows cement paste fluidity changes over time, which is mixed with BTC100 and BTC300 under the same conditions. Figure 2 shows the change of the corresponding water slurry viscosity with time and shows that the initial fluidity of cement paste mixed with BTC300 is 225mm, which is smaller than 240mm of the one mixed with BTC100; paste viscosity that is 555mPa.S is also very big, but half an hour later the fluidity increases to 265mm, and the viscosity decreases to 375 mPa.S. Two to three hours later, the liquidity declines gradually and the viscosity rises more slowly, so the initial dispersion of early strength type polycarboxylate water reducer BTC300 is lower than that of ordinary polycarboxylate water reducers BTC100, and it has a scattered release effect, of which the dispersion effect is better.

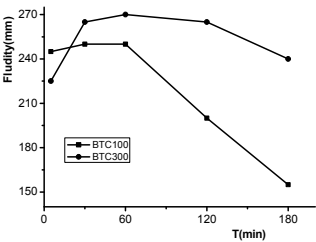


Figure 1. Cement paste fluidity changes with time.

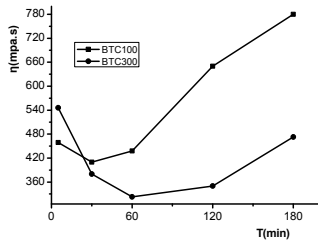


Figure 2. Cement slurry viscosity changes with time.

Influence of polycarboxylate water reducer on water reduction rate. The relationship between content and water reduction rate of ordinary BTC100 and early strength BTC300 is basically similar (Figure 3). The relationship between content and water reduction rate is divided into three proportional sections. For BTC300 water reducer, the relation is a linear relationship under amount of 0.1–0.25%, 0.25–0.55%, and 0.55–1.05%, of which the slope becomes smaller. From 0.55–1.05% ash content conditions, there is a linear relationship with water reducing rate and the slope becomes smaller, but for BTC100 water reducer, the relationship is a similar linear relationship superplasticizer in 0.1–0.2%, 0.2–0.50%, and 0.50–1.05% at a dosage of a linear relationship in the amounts of 0.1–0.2%, 0.2–0.50%, and 0.50–1.05%. Under conditions of a low amount, 0.2%, the water reduction rate of BTC100 is higher than that of BTC300, and later when the content reaches 1.0%, the water reduction rate of BTC300 water reducer can reach more than 40%, far higher than 37.0% of BTC100 water reducer.

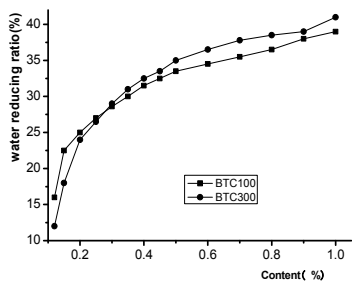


Figure 3. Water reduction ratio with content of BTC100 and BTC300.

Mechanical properties of high performance C30 ~ C80 precast concretes. Both ordinary BTC100 and early strength BTC300 water reducers can be used for the preparation of C30 ~ C80 precast concrete with a high-performance, 28-day strength of concrete mixed with BTC300 and is slightly higher than the corresponding one mixed with BTC100. As can be seen from Figure 4, the early compressive strength of C30-C50 concrete mixed with ordinary BTC100 or early strength BTC300 water reducer of 1–3 days was significantly different under normal temperature conditions; the former has lower strength and the difference becomes larger as the strength grade increases. For C80 concrete (Figure 5), under conditions of 10°C, the 1-day strength 51.6MPa of concrete mixed with BTC300 is more than 50% larger than the 1-day strength 35.0MPa of concrete mixed with BTC100, while under conditions of a temperature of 30°C, the 1-day strength 56.6MPa of concrete mixed with BTC300 is much larger than the 1-day strength 43.2MPa of concrete mixed with BTC100. Because steam curing is often used to improve early strength of precast concrete, the same C40 concrete sets mixed with BTC100 and BTC300 are used for comparison. As shown in Figure 6, concrete mixed with BTC300 has the higher strength after steam curing with mold, reaching 28.9 MPa, which is far greater than 1-day strength 16.2MPa at the normal temperature (Figure 4). Analysis indicates that when the environment temperature is low, due to the lower side chain density of ordinary BTC100 water reducer, the main chain has a higher content of carboxylic acid groups, which retards cement hydration to a certain extent, resulting in its slow early strength development; but under a high environment temperature, the gap of enhancement between two different superplasticizers is not that obvious.

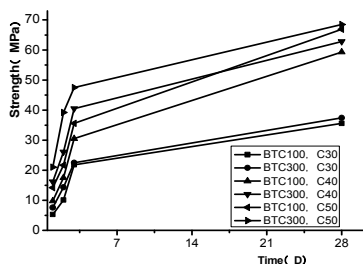


Figure 4. C30 ~ C50 concrete strength instars.

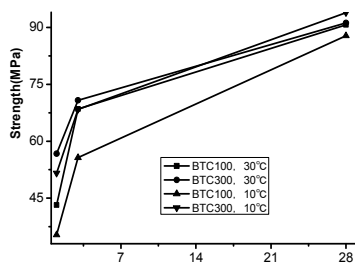


Figure 5. C80 concrete strength instars.

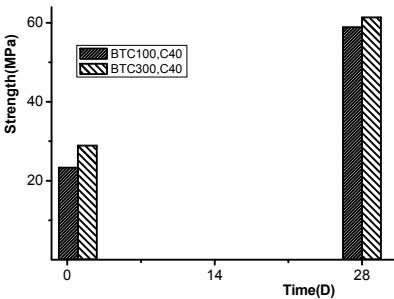


Figure 6. Strength of C40 concrete after 1 day and 28 days of steam curing.

Probe into the mechanism of early strength polycarboxylate water reducer.

Figures 7 and 8 show that the average molecular weights of BTC100 and BTC300 are similar. The distribution of BTC300 molecular weight is narrow and presents a normal distribution, the number of surplus small monomer polymers whose molecular weight is less than 1000 is minimal is small, and the average molecular weight of BTC100 is so large that there exists large number of monomer polymer, which has a bad effect on the early strength of concrete. From the synthesis process and molecular weight distribution of water reducer to analyze, super early strength superplasticizer BTC300 using synthetic technology at normal temperature is different from superplasticizer BTC100 using heating synthesis technology; low reaction temperature is beneficial for adequate reaction between monomers. As for molecular design, a macro monomer of BTC300 is a mix of methallyl polyethylene glycol and isopentenyl polyethylene glycol. Because macro monomer of BTC100 is for a single use, the molar ratio of small monomer and macro monomer of BTC300 is smaller.

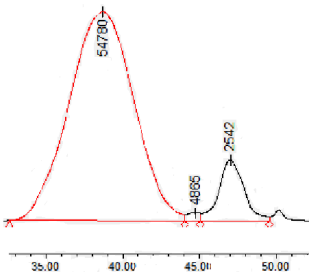


Figure 7. GPC analysis of BTC300.

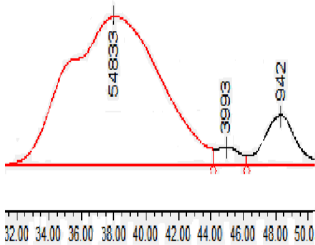


Figure 8. GPC analysis of BTC100.

A higher density of side chain molecules of water reducer improves the adsorption state of copolymer molecules on the surface of cement particles, thus contributing to the growth in strength. Figure 9 is a rate curve of the cement hydration heat releasing, which shows that early strength BTC300 significantly accelerates cement hydration. The time of maximum hydration heat rate is 10 hours before adding ordinary water

reducer BTC100. As amides and sulfonic acid group are brought into the super early strength BTC300 molecules, both of which can improve the early strength of concrete, the density of macro monomer in the structure is relatively higher with better water absorbing effect. BTC300 has the same super water reduction rate with a larger amount. The powder particles are able to settle better to be dense and release excess water from the inside out so that the early strength can grow rapidly. Due to excessive carboxylate, the uniformity of ordinary BTC100 molecular structure declines and uneven adsorption affects further compacting of cement particles and cannot release excess water. It can be seen that the early strength mechanism of BTC300 lies in its relatively uniform molecular structure. The amount of active ingredient for high early strength is large, and the number of small molecule polymer is small.

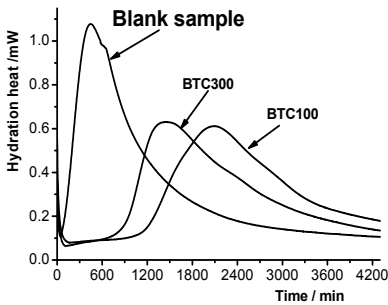


Figure 9. Hydration heat rate curve of cement mixed with superplasticizer.

CONCLUSIONS

(1) The synthesis method of early strength BTC300 superplasticizer differs from that of ordinary BTC100 superplasticizer; normal temperature synthesis takes the place of heating synthetic, which makes the reaction more sufficient and has more molecules active ingredients. With the introduction of the amide groups and sulfonic acid groups, the early strength of concrete can be significantly improved. The reason early strength BTC300 has better water absorption effects may be due to the higher side chain density of the macro monomer, so the post-hydration of concrete is supplemented, and the strength is higher.

(2) Cement paste performance tests show that BTC300 has a slow-releasing and slumping effect, which has lower liquidity and greater viscosity. The liquidity of cement paste mixed with BTC300 is much higher than that of cement paste mixed with BTC100 after 30 minutes, and the viscosity decreases.

(3) Concrete tests show that the water reduction rate of BTC300 is slightly lower than that of BTC100 with the same lower content; but with the same higher content, the water reduction rate of BTC300 is higher. In the content range of 0.2–0.5%, BTC300 and BTC100 have nearly the same water reduction rate.

(4) As for the C30 ~ C80 precast concrete mixed with two different polycarboxylate water reducers separately, whatever the curing method, the early strength of concrete mixed with BTC300 is significantly higher; the early strength

effect is more pronounced as concrete strength improves. Under winter temperature conditions, the 1-day strength of C80 concrete can be higher by 50%, and the average compressive strength of 28 days is also slightly higher.

(5) GPC analysis shows that molecular weight distribution of superplasticizer molecules BTC300 is more in accordance with normal distribution, the number of small monomer polymer in solution that influences early strength of concrete is small, hydration heat test proved cement mixed with BTC300 hydrates faster, of which The second hydration exothermic peak is nearly 10 hours earlier than that of the BTC100, which proves the relatively strong early strength of BTC300.

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Research on Local Buckling Behavior of CFT Circular Columns

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ABSTRACT

Under seismic action, local buckling in plastic hinge regions of CFT columns could attribute to induced force and unrecoverable deformation. To study local buckling behaviors of CFT columns, a theoretical analysis formula of buckling stress through energy principle was proposed, which included elastic buckling stress and inelastic stress. It was found that local buckling of CFT columns are greatly affected by the yielded elasticity modulus of steel tube. An extensive set of tests on forty-two CFT columns under axial force for local buckling behaviors is presented by taking into account of the parameters of slenderness ratio, diameter to thickness ratio, and compressive strength of unconfined concrete. Comparing experimental and theoretical results, the characteristic of yielded elasticity modulus of steel tube was obtained.

INTRODUCTION

In order to achieve the requirement of seismic performance, concrete filled steel tube (CFST) column have been more widely applied in large high-rise or super high-rise modern buildings and long span bridges. With its seismic performance fully embodied, the application of CFST members has been successful to start and become more widespread in developed countries such as Europe, America, and Japan etc. In recent years, China began to use a large number of CFST members in many buildings. Moreover, all of the CFST members nearly went into use in large high-rise or super high-rise buildings. When the earthquake strike the building, the local buckling of CFST column will occur under the action of horizontal earthquake forces, which leads to instability and failure of the member, then to the local or total destruction. Therefore, it is important to investigate the mechanism and the influencing factors of CFST column local buckling.

With a great quantity of studies focused on CFST column, Brian. Uy and Q. Q. Liang presented finite element analysis method to predict the local buckling stress of concrete filled thin-walled steel tube box column and the behavior of post-local buckling. The critical buckling stress formula and the effective width formulas they proposed were based on experience results relatively. Ge Hanbin et al. investigated the strength and deformation of concrete filled thin-walled steel tube box column. It was found that the width thickness ratio and stiffness of the stiffener inside had a great influence on mechanical properties of column. Mark. A. Bradford et al. presented energy method to study the local buckling of concrete filled steel circular

tube columns under axial compression, and then offered the critical buckling stress formula of steel tube compared to the empty ones. Wang Kaining, based on two kinds of models of the foundation strut and the cylindrical shell, solved the critical buckling stress of concrete filled circular steel tube column subjected to axial compression. The upper limit value was gained. According to these, it is known that there have been various studies on CFST column, most of which were under axial compression. On one side, the comparison was made from experimental results and theoretical derivation combined with the research of CFST rectangular columns buckling. On the other side, the studies on buckling of concrete filled circular steel tube are still in the theoretical stage, without comparison to the experimental results formally. Therefore, this paper presents the energy method to predict theoretical analysis and formula deduction of concrete filled circular cross-section steel tube under axial compression. Influence of slenderness ratio, diameter to thickness ratio, concrete strength and other factor on the buckling stress, and the critical buckling stress formula derived in the computational model were also analyzed.

COMPUTATION MODEL SELECTION AND FORMULA DEDUCTION

Computation Model Selection

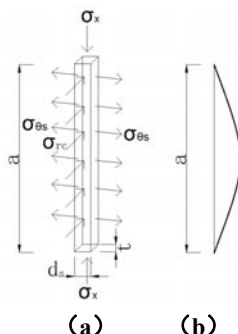


Figure 1. Stress model and buckling deformation

The stress model of the steel tube as shown in Fig. 1 (a), taking a rectangle element at the buckling position of the steel tube. a is the length of the element, width is d_s , and the thickness of steel tube is t . In the calculation, the element subjected to the axial compressive stress (σ_x), the hoop stress ($\sigma_{\theta s}$) and the radial stress of the concrete (σ_{rc}). Buckling mode of the steel tube is shown in Fig. 1(b).

Local Buckling Stress Calculation

Mark. A. Bradford et al. have presented energy method for calculating the local buckling stress of circular steel tube under axial compression and the basic assumptions should be needed as follows: (1) The test members of CFST is the ideal uniform bar; (2) There is no slip between steel tube and concrete in the loading process; (3) Deflection curve of the buckling tube is sinusoidal; (4) Since concrete is regarded as rigid foundation, the radial force is so small that it can be ignored in the calculation; (5) With the end part of column subjected to the axial compressive load, the local buckling of steel tube take place. The calculation formula of elastic buckling stress given by Mark. A. Bradford et al. as

$$\sigma_{xcr} = \frac{p_x}{t} = \frac{2E_s t}{R\sqrt{1-\mu^2}} \tag{1}$$

Where E_s elastic modulus of steel tube, R is is the steel tube outer diameter;

It can be found from Eq. (1) that in elastic range the buckling stress (σ_{xcr}) should be only regard to the diameter to thickness ratio of column (R/t). σ_{xcr} is inversely proportional to R/t and σ_{xcr} can be bigger as R/t is relatively less. So that σ_{xcr} may exceed the yield strength. Therefore, not all local buckling of CFST columns occurred in the elastic state when elastic buckling stress is more than the yield stress of steel tube, which can illustrate that the buckling of steel tube should occur in inelastic region. In this process that steel tube has entered the elastic-plastic stage, tangent modulus (E_s') of steel tube buckling in elastic-plastic state need be used instead of the elastic modulus. The buckling stress of local buckling in the elastic-plastic range of steel is given by

$$\sigma_{xcr} = \frac{p_{x'}}{t} = \frac{2E_s' t}{R\sqrt{1-\mu^2}} \tag{2}$$

Where E_s' is tangent modulus of steel tube buckling in elastic-plastic state.

NUMERICAL SIMULATION

From Eq. (1), the buckling stress formula, we can obtain the relationship between the bucking stress of steel tube and the elastic modulus or diameter to thickness ratio, with the figure shown as follows:

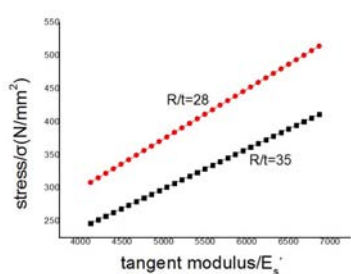


Figure 2.The influence of tangent modulus

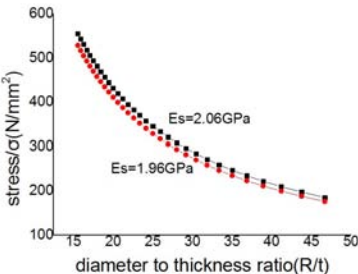


Figure 3. The influence of thickness ratio

As show in Fig. 2, the selection of tangent modulus has a great effect on the calculation results of buckling stress when the bucking occurs. With the increase of tangent modulus, buckling stress also increases. Fig. 3 shows curve of the relationship between the buckling stress and diameter to thickness ratio, and with the increase of diameter to thickness ratio, the buckling stress of steel tube decreases. Because of the less fluctuation of the elastic modulus of steel at room temperature, around 196GPa to 206GPa, the impact on the buckling stress is small.

NUMERICAL ANALYSIS TEST

Component size

Considering three factors: the core concrete strength, diameter to thickness ratio, slenderness ratio, we carried out the tests on 14 groups of CFST circular cross-section column under axial compression, and studied on failure modes of CFST column of local buckling and post buckling. The parameters of specimens are listed in Table 1:

Table 1. Parameters of Text Specimens

	$f_c = 46MPa$, $L=450mm$					$f_c = 52MPa$ $L=450mm$			$f_c = 63MPa$ $L=450mm$			$f_c = 46MPa$ $L=750mm$		
	Z-01	Z-02	Z-03	Z-04	Z-05	Z-06	Z-07	Z-08	Z-09	Z-10	Z-11	Z-12	Z-13	Z-14
R	140	140	140	108	180	140	140	140	140	140	140	140	140	140
t	4	5	6	6	6	4	5	6	4	5	6	4	5	6
R/t	35	28	23	18	30	35	28	23	35	28	23	35	28	23
L/R	3.2	3.2	3.2	4.2	2.5	3.2	3.2	3.2	3.2	3.2	3.2	5.4	5.4	5.4

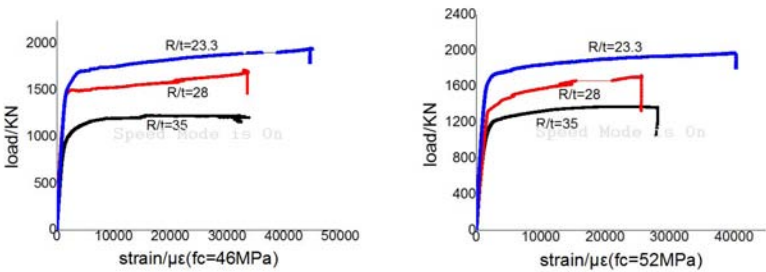
Where R is the steel tube outer diameter, t is the wall thickness of steel tube, R/t is the diameter to thickness ratio of steel tube, L/R is the slenderness ratio of column; the tests tubes are all Q235 seamless steel tubes.

Failure process of components

At the beginning of loading, the shape of specimens in elastic stage did not have obvious change. When the load exceeded the ultimate load of 50% approximately, tube went gradually into the elastic-plastic stage with steel tube rust falling off on the surface. As the load increased, the load-strain curve deviated from the straight line, presenting the plastic properties, and the load-strain curve tended to be flat. When the load exceeded the ultimate load of 85%~90%, there was local convex phenomenon of steel tube in the middle or the end region. At this moment, the local buckling of steel took place and the concrete was crushed, but because of restrained ring effect on concrete caused by steel tube and stability from core concrete to steel tube wall, the specimens could still continue to bear the load. After exceeding the ultimate bearing capacity, load-strain curve fell down and bearing capacity of column decreased. The final member could damage caused by the large convex, the displacement of oblique shear or the large amount of longitudinal compression.

Experiment Result Analysis

As shown in figures below, it's the effect of influencing factors of CFST specimens on the load-stress curve.



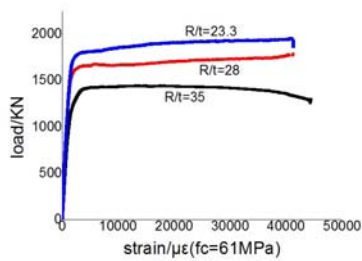


Figure 4. The influence of diameter to thickness ratio

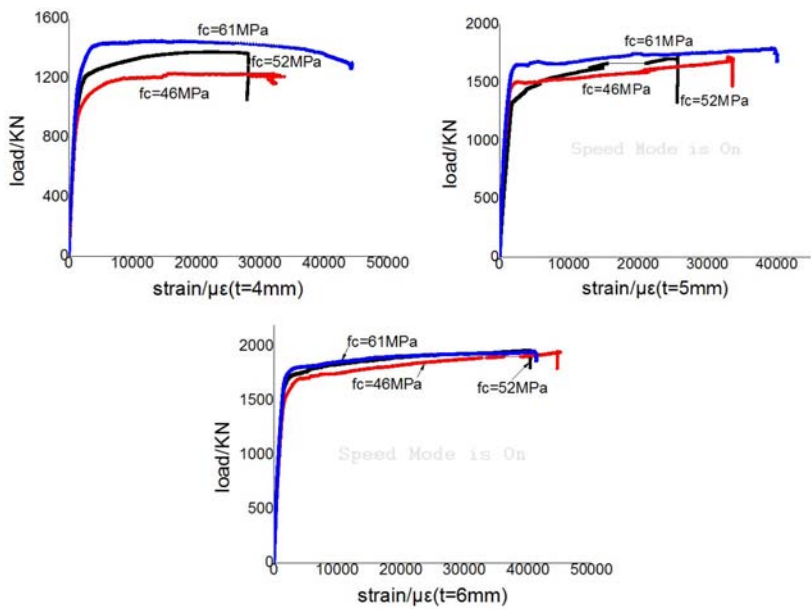
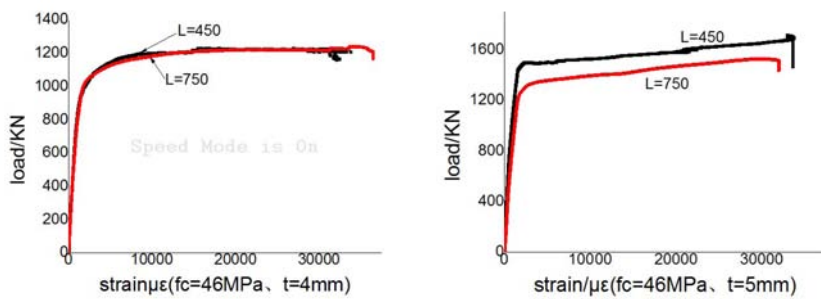


Figure 5. The influence of core concrete strength



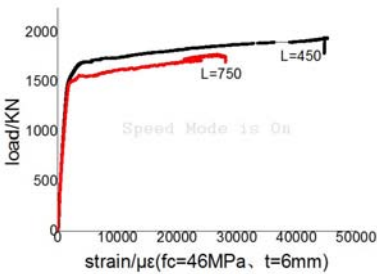


Figure 6. The influence of slenderness ratio

The influence of diameter to thickness ratio on bearing capacity

As shown in Fig. 4 above, the diameter to thickness ratio of column section has an important influence on bearing capacity, With the outer diameter of column section remaining the same and the thickness of tube wall changing, the bearing capacity increases as the diameter thickness ratio decreases and exceeds to 35, 28.3 and 23.3 respectively.

The influence of core concrete strength on bearing capacity

As shown in Fig. 5 above, it's given the load-stress curve of the influence of concrete compressive strength. With the difference of core concrete compressive strength, the influence on the load-stress curve is not much. With the increase of core concrete compressive strength, the bearing capacity of columns increases to some degree, but the influence of concrete compressive strength on the percentage of the bearing capacity of members is low relatively.

The influence of slenderness ratio on bearing capacity

As shown in Fig. 6 above, the bearing capacity of specimens with a big slenderness ratio is little smaller than the bearing capacity of specimens with small ones. Observed in the whole test process, for the member of the larger slenderness ($L/R=5.4$) under axial compression, there was the lateral deflection and the bending deformation with the increase of load. At the moment, there was a phenomenon of steel tube buckling in the middle span of the compression side. It was found that the buckling took place because of the pinching caused by the bending, not the local buckling subjected to axial loads. When the deflection of the middle span reached a critical value, the load began to decline and the deformation increases rapidly, finally members could lose stability.

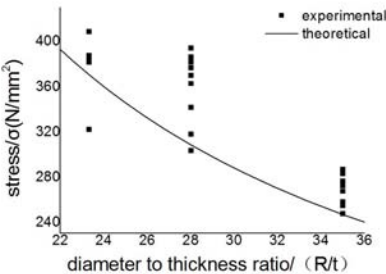


Figure 7. The influence of diameter to thickness ratio

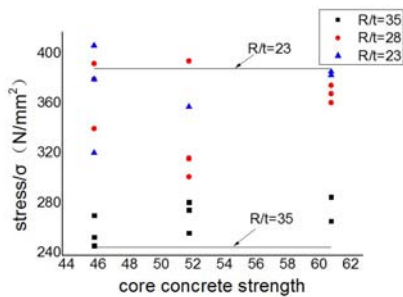


Figure 8. The influence of core concrete strength ratio

For the specimens with relatively large slenderness ratio ($L/R=5.4$), the failure form is different from other ones and can be regarded as the one of a member under axial compression. Therefore, calculation of the buckling stress is given by the equation

$$\sigma_{cr} = \frac{P_{cr}}{A} \tag{3}$$

The influence of slenderness ratio on buckling stress as shown in Fig.9:

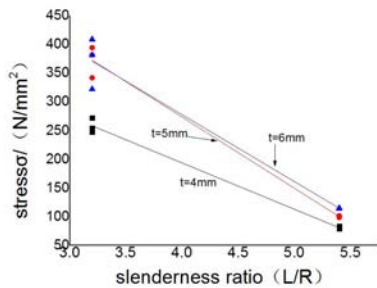


Figure 9. The influence of slenderness ratio

The influence of diameter to thickness ratio on buckling stress

As shown in Fig. 7 above, it's the influence of the diameter to thickness ratio on buckling stress. Compared with the experimental value and the theoretical curve, the buckling stress decreases as the diameter thickness ratio increases with the outer diameter of column section remaining the same.

The influence of core concrete strength on buckling stress

As shown in Fig. 8 above, it's given the load-stress curve of the influence of concrete compressive strength on buckling stress. The diameter to thickness ratios of the two curves of theoretical value of buckling stress are 23 and 35 respectively. As can be seen, the specimens buckling stress of the diameter to thickness ratio around 23 to 35 are almost within this range, that's it, with the change of compressive strength of core concrete, the fluctuation is very small for the specimens with different diameter to thickness ratio. The experimental results also verify the calculation formula of buckling stress that the buckling stress has nothing to do with

concrete strength.

The influence of slenderness ratio on buckling stress

As shown in Fig. 9 above, it's given the load-stress curve of the influence of slenderness ratio on buckling stress, that's it, with the increase of slenderness ratio, the buckling stress of CFST column decreases fast. Because for the specimens with larger slenderness ratio under axial compression, the bending deformation on the pillar caused by the lateral deflection take place, which can lead to instability. The instability mode belongs to elastic instability of compressive and flexural members, differing from the specimen instability with smaller slenderness ratio caused by local buckling of the steel tube wall.

The selection of tangent modulus

Through theoretical analysis and the calculation of the test results, it was found that ratio of tangent modulus and elastic modulus of steel tube are shown in Table 2 as the local buckling of the steel tube occurs. When the slenderness ratio of the specimens is 5.4, there is the lateral deflection failure instead of local buckling failure, so the specimens Z-12 ~ Z14 is not listed in Table 2. Confirmatory analysis showed that the tangent modulus of steel tube was $\frac{1}{30}E_s$ approximately when the local buckling of the CFST column took place under axial compression. The theoretical values were good agreement with the experimental values.

Table 2. The comparison between the buckling stress calculation and the test

		Tangent modulus E_s' / E_s	Experimental value(σ_e)	Calculated value(σ_c)	σ_e / σ_c
Z-01	(1)	0.018	272.34	246.80	1.10
	(2)	0.023	254.68	246.80	1.03
	(3)	0.022	247.76	246.80	1.00
Z-02	(1)	0.014	394.34	308.50	1.28
	(2)	0.015	341.92	308.50	1.11
	(3)	0.016	381.73	308.50	1.24
Z-03	(1)	0.014	408.93	370.19	1.10
	(2)	0.017	381.6	370.19	1.03
	(3)	0.018	322.27	370.19	0.87
Z-04	(1)	0.036	288.09	465.90	0.62
	(2)	0.042	349	465.90	0.75
	(3)				
Z-05	(1)	0.017	353.06	287.93	1.23
	(2)	0.026	386.6	287.93	1.34
	(3)	0.023	386.71	287.93	1.34
Z-06	(1)	0.022	276.58	246.80	1.12
	(2)	0.029	258.11	246.80	1.05
	(3)	0.030	283	246.80	1.15
Z-07	(1)	0.014	386.43	308.50	1.25
	(2)	0.021	318.02	308.50	1.03
	(3)	0.010	303.51	308.50	0.98
Z-08	(1)				

Z-09	(2)				
	(3)	0.011	382.72	370.19	1.03
	(1)	0.012	267.62	246.80	1.24
	(2)	0.017	287.07	246.80	1.16
	(3)				
Z-10	(1)	0.008	376.83	308.50	1.22
	(2)	0.011	362.79	308.50	1.18
	(3)	0.016	370.06	308.50	1.20
Z-11	(1)	0.009	387.75	370.19	1.05
	(2)	0.014	384.88	370.19	1.04
	(3)	0.019	384.84	370.19	1.04

Where the average value of tangent modulus E_s' is $0.019E_s$ and the average value of $\sigma\epsilon/\sigma_c$ is 1.096.

CONCLUSION

The theoretical analysis and the experimental results showed that the local buckling of steel tube cannot occur in the elastic state generally, but in elastic-plastic state. Through the analysis of test results, the tangent modulus was about $\frac{1}{50}E_s$. Substituting into the buckling stress calculation formula and then compared to experimental results, both of them agree well with each other. First, there was a lateral deflection and a bending deformation for CFST columns of large slenderness under axial compression. Then the buckling occurred because of the pinching caused by the bending on the compression side. Finally, the excessive bending deformation caused the failure of CFST column. The failure way above was different from other components of smaller slenderness ratio because the failure of local buckling was under axial compression. Furthermore, tangent modulus might not be the same because different slenderness ratio and diameter to thickness ratio may effect on the restraint coefficient, which was needed for continue study deeply.

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Predicting Dynamic Modulus of Florida Hot Mix Asphalt Mixtures

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ABSTRACT

The dynamic modulus is one of the fundamental properties defining the response of asphalt mixtures in a flexible pavement system. It is an essential input of the Mechanistic-Empirical Pavement Design Guide (MEPDG) software for asphalt pavement design. The dynamic modulus can be evaluated by several existing direct testing procedures, but the tests are difficult and costly, proving complicated for engineers to carry out during the pavement design process. Many studies have been done concerning the predictive model of dynamic modulus. This presentation presents a comprehensive study about developing a predictive model of dynamic modulus for characterizing the asphalt concrete mixtures used in Florida. A laboratory experimental program was developed to evaluate the dynamic modulus of selected Florida Superpave asphalt concrete mixtures. Based on the test results and analyses, a dynamic modulus prediction model was constructed. The results showed that the variables related to aggregate gradation, mix volumetric, percent weight of asphalt content, loading frequency and temperature had influence on the dynamic modulus of the hot mix asphalt mixtures. The predicted results of the developed model were very comparable with the test data. The predictive model could be more suitable for predicting dynamic modulus of the hot mix asphalt mixtures used in Florida.

INTRODUCTION

The dynamic modulus of asphalt concrete is determined as the ratio of the amplitude of applied stress to the amplitude of measured strain response experimentally. It is a material response parameter that determines the strain and displacements of pavement structure. The Mechanistic-Empirical Pavement Design Guide (MEPDG) developed under the National Cooperative Highway Research Program (NCHRP) Project 1-37A

uses dynamic modulus (E^*) as the design stiffness parameter at all three levels of hierarchical input for the hot mix asphalt (HMA) characterization (NCHRP 2004). The dynamic modulus should be the most important parameters in the pavement design.

Many studies have been done about the predictive model of dynamic modulus. The Witczak E^* model remains the most widely used model for characterizing the dynamic modulus of asphalt concrete mixtures (Witczak *et al.* 1996; Andrei *et al.* 1999; Bari *et al.* 2006). The later version of the Witczak E^* predictive equation was based on this database (known as the “UMD E^* Database”) that contains 2750 test data points from 205 AC mixtures during this period. Bari and Witczak (Bari *et al.* 2006) presented their study about the predictive models, which contained E^* testing on 346 AC mixtures that provided 7400 E^* test data points. They developed a revised E^* predictive model equation that directly linked the dynamic modulus of HMA mixture with the complex shear modulus of the binder used in the mixture. There were also some other models used to predict the dynamic modulus of asphalt concrete (Buttlar *et al.* 1999, Christensen *et al.* 2003, Collop *et al.* 2004, Shu and Huang 2008). All of these models provided some reasonable approaches for estimating the dynamic modulus of asphalt concrete.

This paper presents a comprehensive study about developing a simplified predicting model of dynamic modulus for characterizing Florida asphalt concrete mixtures. A laboratory experimental program was developed to evaluate the dynamic modulus of 20 selected Florida Superpave asphalt concrete mixtures.

EXPERIMENTAL PROGRAM

The laboratory dynamic modulus tests involved 20 selected Florida Superpave asphalt concrete mixtures with a range of aggregates and mix designs (Ping *et al.* 2007). One type of asphalt binder, PG 67-22 (AC-30), the only asphalt used in Florida, was applied for all the mixture tests. The primary objective of the test program was to set up a database of the dynamic modulus of asphalt concrete mixtures used in Florida from the dynamic modulus test (DMT).

In general, for each test series, three DMT specimens were tested to determine the dynamic modulus of each mix design. Test specimens with dimensions of 150 mm (5.9 in.) in diameter by 165 mm (6.5 in.) in height were first prepared. Specimens were with approximately 4.0 percent air voids following the design air voids requirement in the Superpave mix design criteria. These mix designs were selected as they are commonly used Florida Department of Transportation (FDOT) gradations and are known to perform well in the field. The nominal maximum aggregate sizes for the mixtures tested are 19.0 mm, 12.5 mm, and 9.5 mm. The gradation curve should go through between the upper and lower control points specified by Superpave. The nominal 100 mm (4.0 in.) diameter test specimens were cored from the center of the gyratory specimens and were subsequently cut to 150 mm (5.9 in.) in height. The dynamic modulus and phase angles were measured by applying compressive sinusoidal (haversine) load generally following the Simple Performance Test System developed in NCHRP Project 9-29 (Figure1). The deformations were measured through three Linear Variable Differential Transducers (LVDTs) (Figure1).

Tests were performed at temperatures of 5 °C, 25 °C and 40 °C and at frequencies of 25 Hz, 10 Hz, 5 Hz, 1 Hz and 0.5Hz to simulate moving wheel loads. The test temperatures were within the range of temperature variations generally expected for the pavements in Florida. Testing began with the lowest temperature and proceeded to the highest temperature. At a given temperature level, the testing began with the highest frequency of loading and proceeded to the lowest. This temperature-frequency sequence was carried out to cause minimum damage to the specimens before the next sequential test.

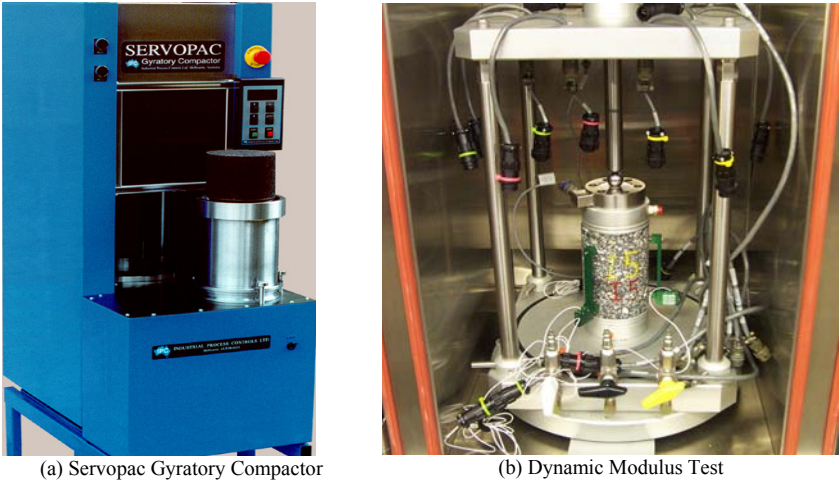


Figure 1. Dynamic Modulus Test Setup

For each load frequency and testing temperature, one test data file was obtained, in which the dynamic modulus and phase angle in degrees were computed. After all the dynamic modulus and phase angle values were calculated for each specimen under the same test conditions, the average values for both parameters were calculated. In addition, the procedures developed by Pellinen and Witczak (Pellinen and Witczak 2002) were used for obtaining the predicted master curve of all the mixtures in the study.

DEVELOPMENT OF SIMPLIFIED MODEL

Dynamic modulus master curve. Pellinen and Witczak (Pellinen and Witczak 2002) developed a mathematical model by a sigmoidal fitting function for master curve construction. The shift can be calculated by solving the shift factors simultaneously with the coefficients of the sigmoidal function:

$$\log(| E^* |) = \delta + \frac{\alpha}{1 + e^{\beta + \gamma \log f_r}} \tag{1}$$

Where, $\log(| E^* |)$ = log of dynamic modulus

- f_r = reduced frequency
 δ = minimum modulus value
 α = span of modulus value
 β, γ = shape parameters

Development of simplified model. The Witczak dynamic modulus predictive model is the most popularly used model for characterizing the dynamic modulus of HMA mixtures. A simplified predictive model was constructed based on the Witczak E* model according to the dynamic modulus test results in this study for the Florida asphalt concrete mixtures. The modeling analysis was aimed at reducing the error resulting from prediction by comparing the predicted values with the observed values for the same values of the input variables in different ways. Coefficient of determination (R^2) was used to find out the goodness of fit. A good model would have a high R^2 (close to unity).

From the test results and analysis above, HMA mix behavior was largely reflected by the gradation (such as P_{200} , P_8 , P_4 , $P_{3/8}$ etc.) and voids characteristics (such as P_b and VFA etc.) of mixture. Furthermore, the PG 67-22 is the only type of asphalt binder used in Florida, so the binder is the same; for viscosity, the different affecting parameters are only the test temperature and frequency. Here the temperature and frequency can be used as a function for replacing the viscosity of binder, because they are the direct factors for the dynamic modulus of asphalt concrete test results; it should be more accurate to input the temperature and frequency directly. According to the test results and the previous predicting models, and taking into account these direct and indirect interrelationships, the important predictor variables for the mix $|E^*|$ were primarily listed as follows:

- Aggregate gradation (P_{200} , P_8 , P_4 and $P_{3/8}$)
- Percent weight of asphalt (P_b)
- Voids filled with asphalt (VFA)
- Loading frequency (f)
- Test temperature (T)

A number of candidate E* models were developed and evaluated in the course of this research. Each model was tested for rationality, accuracy, precision, bias, trend, sensitivity and overall performance. Among all models evaluated, the following model turned out to be the most promising one (the correlation coefficient $R^2=0.957$):

$$\log |E^*| = 2.312 + 0.01P_{200} + 0.01P_8 - 0.013P_4 - 0.002P_{3/8} + 0.024P_b - 0.043VFA + \frac{[-1.34 - 0.019P_8 + 0.022P_4 + 0.004P_{3/8} - 0.055P_b + 0.052VFA]}{1 + e^{(-8.267 - 0.722\log f + 5.397\log T)}} \quad [2]$$

Where, $|E^*|$ = dynamic modulus, in 10^5 psi

T = Test temperature, in $^{\circ}\text{C}$

f = load frequency, in Hz

VFA = Voids filled with asphalt, % by volume

- P_b = Percent weight of asphalt, % by weight
- $P_{3/8}$ = cumulative percent retained on 3/8 in (9.5mm) sieve, % by weight
- P_4 = cumulative percent retained on No. 4 (4.75mm) sieve, % by weight
- P_8 = cumulative percent retained on No. 8 (2.36mm) sieve, % by weight
- P_{200} = percent passing on No. 200 (0.075mm) sieve, % by weight

EVALUATION OF SIMPLIFIED MODEL

Figure 2 is a plot of observed E^* stiffness data from the DMT test results versus the predicted E^* stiffness data from the simplified predicting model under the same input conditions. The plot shows that all the individual data points are around the line of equality without any trace of significant bias, and the linear fitting result is $y=0.9504x$. The linear fitting parameter is 0.9504, near to 1, and there is a very good correlation coefficient ($R^2=0.957$). This is a very small level of bias, which can satisfy the requirements of the engineering projects.

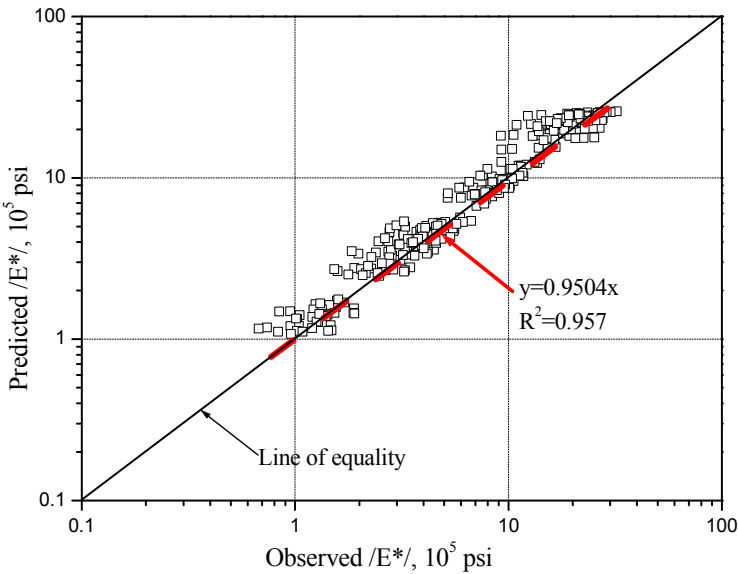


Figure 2. Comparison between Predicted and Observed E^* (1 Psi=6.89 Kpa)

CONCLUSION

A laboratory experimental program was developed to evaluate 20 selected Superpave asphalt concrete mixtures with a range of aggregates and mix designs. All mixtures were tested for dynamic complex modulus. Based on the test results, analyses, and findings of this study, a simplified E^* model was constructed, taking into account both the test results and previous models. The findings and conclusions may be drawn as follows:

- (1) The predicted E^* stiffness data is very close to the testing observed E^* stiffness data; there is a very good correlation and a small level of bias, which can meet the accuracy requirements of the asphalt pavement design.
- (2) The variables related to aggregate gradation, mix volumetric, voids filled with asphalt, loading frequency and temperature have the most influence on the E^* stiffness of HMA mixes.
- (3) This study provides a model for State DOT's to implement the M-E Pavement Design Guide with inputs and calibrations that are specific to their region.

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Applications of Artificial Neural Networks to Pavement Prediction Modeling: A Case Study

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ABSTRACT

Artificial neural networks (ANN) have been used in many pavement prediction modeling analyses. However, the convergence characteristics and model selection guidelines are rarely studied due to the requirement of extensive network training time. Thus, the techniques and applications of back propagation neural networks were briefly reviewed. Three ANN models were developed using deflection databases generated by factorial BISAR runs. A study of the convergence characteristics indicated that the resulting ANN model using all dominating dimensionless parameters was proved to have higher accuracy and require less network training time and data than the other counterpart using purely input parameters. Increasing the complexity of ANN models does not necessarily improve the modeling statistics. With the incorporation of subject-related engineering and statistical knowledge into the modeling process, reasonably good predictions may be achieved with more convincing generalization and explanation yet requiring minimal amount of time and effort.

INTRODUCTION

Predictive models have been widely used in various pavement design procedures, evaluation, rehabilitation, and network management systems. Empirical and mechanistic-empirical approaches using statistical regression techniques have been utilized extensively in predicting extremely complicated pavement responses and performance indicators for more than four decades. Using purely empirical concepts to develop predictive models is not recommended. Lee (1993) proposed a systematic statistical and engineering modeling approach which strongly recommends to incorporate theoretical engineering knowledge, expert experience, heuristics, and statistical data analysis and regression techniques altogether into the framework to develop more mechanistic-based predictive models. In addition to the conventional “parametric” linear and nonlinear regression techniques, several ingenious iterative regression techniques in the area of “robust” and “nonparametric” regressions were also incorporated. The proposed approach has been successfully implemented in the development of many purely mechanistic (Lee, 1993; Lee & Darter, 1994) and purely empirical predictive models (Lee & Darter, 1995), as well as the mechanistic -empirical predictive models adopted in the early analyses of LTPP general pavement studies data (Simpson et al., 1993).

Significant progress has been reported in pavement prediction modeling of simulated data using artificial neural networks (ANN). However, the convergence characteristics and model selection guidelines are rarely studied due to the requirement of extensive network training time. As part of continuous research efforts in pavement design and analysis (Lee & Darter, 1994; Lee et al., 2004), the techniques and applications of back propagation neural networks were briefly reviewed. Factorial BISAR runs for different pavement systems are conducted to generate the deflection databases for the analysis. Artificial neural networks were utilized to improve the prediction accuracy of simulated pavement deflections (Liu, 2004). This study strives to illustrate the benefits of incorporating the principles of dimensional analysis, subject-related knowledge, and statistical knowledge into modeling process.

THE CONCEPT OF ARTIFICIAL NEURAL NETWORKS

The concept of ‘neural network’ was originated by the work on ‘perceptrons’ around 1960. They were pictured as networks with a number of inputs x_i and an output (or outputs) y , where the inputs are connected to one or more neurons in the input layer and they are further connected in one or more hidden layers until they reach the output neuron. Artificial Neural Networks (ANN) provides a flexible way to generalize linear regression functions. They are nonlinear regression models but with so many parameters extremely flexible to approximate any smooth function. The most commonly used rule is the generalized delta rule or back propagation algorithm. Ripley (Ripley, 1993) provided the detail definitions and brief derivation of a back propagation network (BPN). The learning procedure has to select the weights and the biases by presenting the training examples in turn several times, while striving to minimize the total squared errors. However, the questions of how many layers and how many neurons should be used were treated very lightly in the literature.

Ripley (1993) also discussed many statistical aspects of neural networks and tested it with several benchmark examples against traditional and modern regression techniques, such as generalized discriminant analysis, projection pursuit regression, local regression, tree-based classification, etc. Ripley concluded that in one sense neural networks are little more than non-linear regression and allied optimization methods. “That two-layer networks can approximate arbitrary continuous functions does not change the validity of more direct approximations such as statistical smoothers, which certainly ‘learn’ very much faster (Ripley, 1993).” Projection pursuit regression highlights the value of differentiated units and other training schemes and offers computation shortcuts through forward and backward selection. Statistical and subject-related knowledge can be used to guide modeling in most real-world problems and so enable much more convincing generalization and explanation, in ways which can never be done by ‘black-box’ learning systems (Ripley, 1993).

BRIEF REVIEW OF ARTIFICIAL NEURAL NETWORKS APPLIED IN PAVEMENT PREDICTION MODELING

Back propagation networks (BPN) can be taught from one data space to another using representative set of data to be learned. The learning process actually refers to

a multi-layered, feed-forward neural network trained by using an error back propagation algorithm or an error minimization technique (Haykin, 1994; Hecht-Nielsen 1990). Ceylan (2004) conducted a literature search summarizing recent ANN applications in pavement structural evaluation such as backcalculating pavement layer moduli and predicting primary pavement responses (e.g., stress and deflection). As with many ANN applications in the literature, original pertinent input parameters were used to generate the training and testing databases. This approach often requires tremendous amount of time and efforts in network training and testing. To reduce the size of the required factorial databases, researchers sometimes opt to fix certain input parameters to some prescribed values as a special case study, which may result in limiting the inference space of the resulting model.

Nevertheless, some earlier ANN literature has also illustrated that the incorporation of the principles of dimensional analysis lead to significant savings during the training set generation. Ioannides et al. (1996) trained a back propagation neural network (BPN) to determine the in situ load transfer efficiency of rigid pavement joints from Falling Weight Deflectometer (FWD) data. Khazanovich and Roesler (1997) developed an ANN-based backcalculation procedure for composite pavements. The multilayer elastic program DIPLOMAT was used to analyze a three-layer pavement system consisting of an AC surface layer over a PCC slab resting on a Winkler foundation. Ioannides et al. (1999) trained BPN models to predict the critical slab bending stress for loading-only, curling-only, and loading-and-curling cases. BPN predictions were compared against the Westergaard closed-form solutions as well as the statistical regression models developed by Lee and Darter (1994) using a small set of factorial data with dimensionless mechanistic variables. It was re-emphasized that mature engineering judgment and in-depth understanding of the mechanics of the phenomenon remain the most reliable guides in the formation of the targeted problems.

Attoh-Okine (1994) proposed the use of ANN models in predicting roughness progression of flexible pavements. Although the results were promising, some built-in functions including learning rate and momentum term which form key neural network algorithm were not investigated. Attoh-Okine (1999) used real pavement condition and traffic data and specific architecture to investigate the effect of learning rate and momentum term on BPN models for predicting flexible pavement performance. Sorsa et al. (1991) indicated that adding many hidden layers gets the network to learn faster and the mean square error becomes a little smaller, but the generalization ability of the network reduces.

BENEFITS OF INCORPORATING STATISTICAL AND SUBJECT-RELATED KNOWLEDGE INTO THE MODELING PROCESS

To illustrate the benefits of incorporating statistical and subject-related knowledge into the modeling process, the following case studies were conducted using a more complicated database (Liu, 2004). A neural network modeling software package called Qnet v2000 for Windows (Vesta Services, Inc., 2000) was adopted for this study.

Development of Flexible Pavement Deflection Databases for the Analysis. Based on the multi-layer elastic theory (Huang, 2004) and the principles of dimensional analysis, the following dominating dimensionless variables were

identified for a three-layer pavement system: E_1/E_2 , E_2/E_3 , h_1/h_2 , and a/h_2 . In which, a is the radius of the applied load, (L); h_1 and h_2 are the thickness of the surface and base layers, (L); E_1 , E_2 , and E_3 are the Young's moduli of the surface layer, base layer, and subgrade, respectively, (FL⁻²). A series of factorial BISAR runs was conducted with the following ranges to cover most practical pavement data: $0.5 \leq E_1/E_2 \leq 170$, $0.5 \leq E_2/E_3 \leq 170$, $0.2 \leq h_1/h_2 \leq 2.4$, and $0.5 \leq a/h_2 \leq 5.0$. A BASIC program written by Dr. Alaeddin Mohseni was used to automatically generate the input files and summarize the results to avoid untraced human errors. A pavement response database including the aforementioned dimensionless variables, deflections at the center of load (D_0), horizontal strain (ϵ_t) and vertical strain (ϵ_v) at the bottom of the surface layer was obtained. A training database with 3,600 data points and an independent testing database with 1,728 data points were used in this study (Liu, 2004).

Development of ANN Models for Flexible Pavements. The training database was randomly separated into 3,400 data points for actual training and the remaining 200 observations for monitoring the training process. Hyperbolic tangent activation function was chosen in this case study (Vesta Services, Inc., 2000). The learning rate was set as 0.01. At the first trial (NET1) as shown in Table 1, no transformation was made on both explanatory and response variables. In which, the dependent variable is the maximum deflection (D_0), whereas the explanatory variables include E_1/E_2 , E_2/E_3 , h_1/h_2 , and a/h_2 . Nevertheless, extreme difficulty was encountered in obtaining reasonable convergence. Several other attempts were also conducted using simply the pertinent input parameters (such as a , h_1 , h_2 , E_1 , E_2 , and E_3) as the explanatory variables, but the results of ANN prediction modeling were even worse.

Based on the basic assumptions of conventional regression techniques that the random errors are mutually uncorrelated and normally distributed with zero mean and constant variance, and additive and independent of the expectation function, it is desirable to check the normality of the response variable. The Box and Cox (1964) transformation procedure was adopted to find the approximate power transformation of the response variable (D_0). The S-PLUS Statistical Analysis Software (Mathsoft, Inc., 1997) was used for this analysis. The maximum likelihood estimator λ of various power transformations (Weisberg, 1985) was approximately 0 indicating that a logarithm transformation was appropriate for D_0 . The normal Q-Q plot which graphically compares the distribution of $\log(D_0)$ to the normal distribution represented by a straight line. This indicates that the logarithm of D_0 is approximate to normally-distributed. In the second trial (NET2), convergence was obtained though the number of learning cycles and modeling time were still very high. The root mean squared (RMS) errors were computed accordingly.

According to general statistical principles or using the alternating conditional expectations (ACE) algorithm (Mathsoft, Inc., 1997; Breiman & Friedman, 1985) together with the Box-Cox power transformation technique proposed by Lee (1993), logarithm transformations of D_0 , E_1/E_2 , and E_2/E_3 were recommended for NET3 model. As shown in Table 1, with more statistical knowledge incorporated into the ANN modeling process, the resulting ANN model was proved to have higher accuracy and less network training time than the other counterpart using purely input parameters. Figure 1(a) and 1(b) depict the network

convergence results for NET2 and NET3 during the training process. The goodness of the prediction of $\log(D_0)$ for NET2 and NET3 were also provided in Figure 1(c) and 1(d) during the testing phase. The goodness of the prediction of D_0 for NET2 and NET3 were also provided in Figure 1(e) and 1(f) during the testing phase. With more statistical knowledge incorporated into the modeling process, the resulting ANN model was proved to have higher accuracy and less network training time than the other counterpart using purely input parameters.

Table 1. Comparison of Three Different ANN Models.

ANN Type	NET1	NET2	NET3
Outputs	D_0	$\text{Log}(D_0)$	$\text{Log}(D_0)$
Inputs	$E_1/E_2, E_2/E_3,$ $h_1/h_2, a/h_2$	$E_1/E_2, E_2/E_3,$ $h_1/h_2, a/h_2$	$\log(E_1/E_2),$ $\log(E_2/E_3), h_1/h_2, a/h_2$
Hidden Layer(s)	3	3	2
Neurons in Each Hidden Layer	20-10-5	15-10-5	12-6
Learning Cycle	Cannot converge	200,000	27,000
Modeling Time	> 24 hrs	10 hrs	26 min
RMS	---	Training: 0.0048 Monitoring: 0.0045	Training: 0.0040 Monitoring: 0.0039

CONCLUSIONS

A case study was conducted to illustrate the benefits of incorporating statistical and subject-related knowledge into pavement prediction modeling process. The resulting ANN model using all dominating dimensionless parameters was proved to have higher accuracy and require less network training time and data than the other counterparts using purely input parameters. Increasing the complexity of ANN models does not necessarily improve the modeling statistics. The results also showed that using higher number of neurons and hidden layers sometimes lead to even worse modeling statistics which was an indication of over training and should be avoided. Statistical and subject-related knowledge can be used to guide modeling in most real-world problems and so enable much more convincing generalization and explanation, in ways which can never be done by ‘black-box’ learning systems (7).

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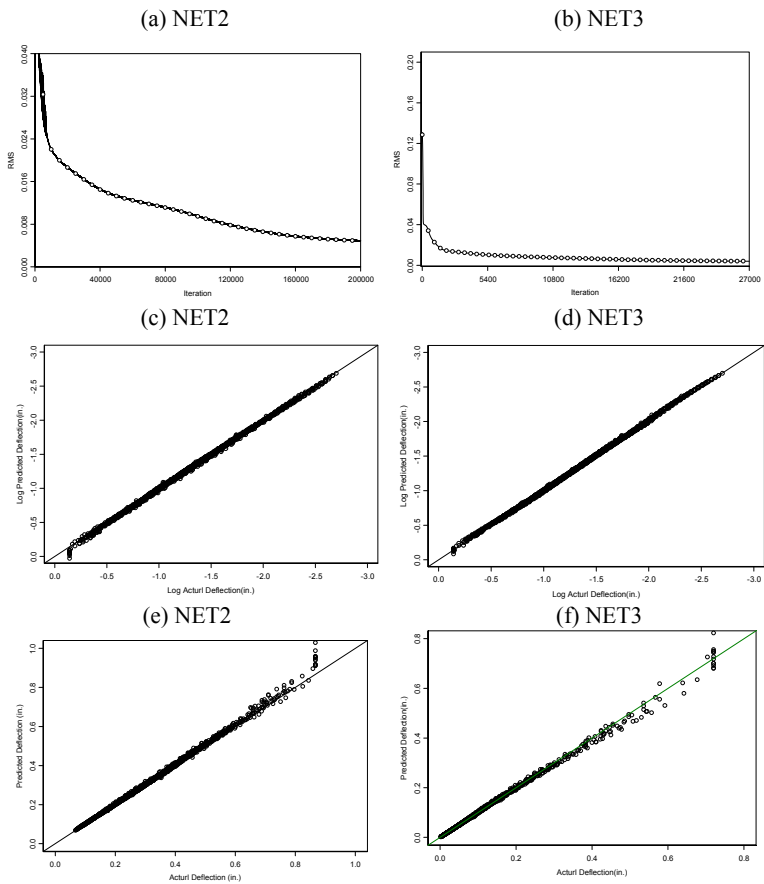


Figure 1. Convergence Results, Goodness of the Prediction of $\text{Log}(D_0)$, and Prediction of D_0 for NET2 and NET3, Respectively.

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Investigation and Analysis for Performance of Cold In-place Recycling Asphalt Pavement in Cold Regions

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ABSTRACT

From the testing of cold in-place recycling asphalt pavement in cold regions of China, combined with pavement design data, some pavement performance indicators and information were obtained, such as design deflection of recycling pavement, actual rebound deflection of pavement before and after regeneration, structure strength coefficient index (SSI), and pavement condition index (PCI). Through the analysis of the test data, condition of the cold in-place recycling asphalt Pavement in cold regions and structural strength improvement of recycling asphalt pavement through regeneration were evaluated. The prediction model of the recycling pavement performance was established. Two approaches to predict the service life of the cold in-place recycling asphalt pavement were developed.

INTRODUCTION

In recent years, with the development of the highway in China, many asphalt roads have been aging gradually. Different levels of distresses have appeared. The first step of the development of Chinese highway, the construction of the highway, is near the end and about to start the second step, the highway network performance monitoring, evaluation, and maintenance. Finding out how to reasonably repair and reconstruct the asphalt pavement becomes an important task in the second step.

Adopting the pavement regeneration technology is a good solution. The regenerated asphalt road has been widely applied due to its economic profit. In this paper, according to the field-collected information and test data, the performance of the recycling asphalt pavement in cold regions was evaluated. According to the data, such as rebound deflection and Pavement Condition Index which are collected on the same route at different time, the renewable pavement performance prediction model was established. Two different methods to predict the length of pavement service life have been developed.

SURVEY OF RECYCLING PAVEMENT CONDITION

In order to study the service condition of the Cold In-place Recycling Asphalt Pavement (CIRAP) in cold regions, the service condition of the CIRAP has been investigated and tested many times in 2008, 2012, and 2013. Large amounts of data have been collected. The information investigated includes: recycling time, recycling method, structural design rebound deflection, structure layer thickness and construction

scheme, etc. The collected data includes: the actual rebound deflection of the pavement surface before recycling, the actual rebound deflection of the road pavement surface after recycling, pavement structure strength index (PSSI), pavement condition index (PCI), and pavement structure strength coefficient index (SSI).

The objects of the investigation are two kinds of recycling pavement structure: cement stabilization of CIRAP recycling for surface and base course together, cement stabilization of CIRAP recycling for base course only.

The first one recycles both the old asphalt surface and the old base. After cold milling, the stabilizer such as cement and lime is added to build recycling base course of the new pavement. And new asphalt concrete will be paved on the recycling base course. This way of recycling is defined as CIRAP recycling for surface and base course together, the structure diagram as shown in Figure 1.

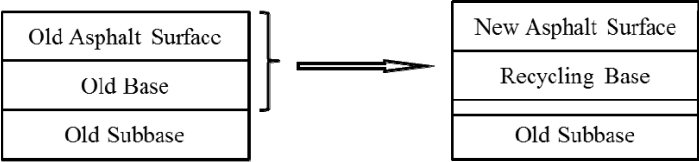


Figure 1. The structure of CIRAP (recycling for surface and base course together)

The second one just recycles the old base course. After cold milling of the surface course, the old surface material is collected or discarded. If collected, it will be recycled via in-factory hot recycling. After cold milling of the base course, the cement stabilization is added to build recycling base course. And the new asphalt concrete or the prepared recycling asphalt concrete by in-factory hot recycling will be paved on the recycling base course. This way of recycling is defined as CIRAP for base course, the structure diagram as shown in Figure 2.

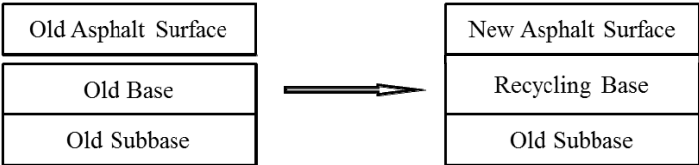


Figure 2. The structure of CIRAP (recycling for base course only)

Based on the two different recycling modes, this paper made the analysis of the performance of CIRAP separately.

ANALYSIS OF STRENGTH IMPROVEMENT CAPACITY OF CIR

SSI which could directly characterize the strength conditions of the pavement is adopted to analyze the strength. Two kinds of SSI are calculated according to the actual rebound deflection on the pavement surface before and after regeneration. As the formulation shown below, the SSI of pavement surface has been calculated (the design deflection is obtained in the part of data collection).

$$SSI = \frac{l_d}{l_0}$$

Where l_d = design rebound deflection of pavement surface;
 l_0 = actual rebound deflection of pavement surface.

The x-coordinate is the SSI increment, every 0.1 as an interval. The percentage of every interval in the total mileage is calculated to draw the bar chart of the probability distribution of SSI increment. The probability distribution curve of the SSI increment shows the capacity of strength improvement and the stability of recycling technology.

For the CIRAP for surface and base, 54 representative road sections are adopted, 50.503 kilometers totally. The situation of SSI improvement is shown in Figure 3.

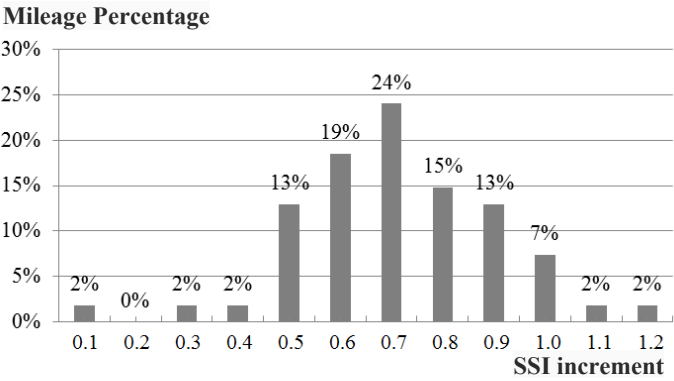


Figure 3. Improvement of SSI for CIRAP recycling in surface and base together

It is clearly reflected by the data in Figure 3 that the SSI increment after recycling presents a law of normal distribution. The expectation is in the vicinity of 0.7, which suggests the SSI is mostly increased by 0.7; and all the data lie between 0.5 and 1.0, only a few quantity of SSI increment under 0.2. Obviously, the technology of CIRAP for surface and base can improve the strength of the pavement effectively and stably.

For the CIRAP for base course, 65 representative road sections are adopted, 63.535 kilometers totally. The probability distribution of SSI increment presents in Figure 4.

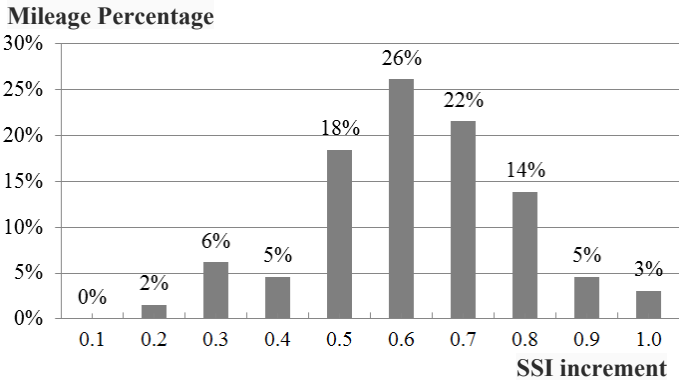


Figure 4. Improvement of SSI for CIRAP recycling in base course only

The data in figure 4 confirmed that the situation of SSI increment after recycling presents a law of normal distribution. The expectation is in the vicinity of 0.6, which suggests the SSI of this kind of recycling pavement is mostly increased by 0.6; and all the data lie between 0.5 and 0.8, only a few quantity of SSI increment under 0.2. Obviously, the technology of the CIRAP for base course can improve the strength of the asphalt pavement effectively and stably.

PAVEMENT PERFORMANCE PREDICTION MODEL

According to the collected data of pavement condition index (PCI), a relationship between time and the change of PCI is established, to evaluate and predict the service condition of the CIRAP. Combined with the data of the same style of recycling and the linear function model, the service life in such recycling mode is calculated as the preliminary forecast service life. In addition, the technology of CIR is evaluated based on the comprehensive analysis above.

According to the variation tendency of PCI, the linear function model of all roads can be written as:

$$PCI = k_i T + b_i$$

where k_i : the gradient of PCI for Road i ; T : the service time of road (in years); b_i : the PCI initial value of i road; i : the serial number of roads, including n roads.

In order to get the most representative straight line, adopting averaging method to get average of all k_i , the calculation formula:

$$k = \frac{1}{n} \sum_{i=1}^n k_i$$

For the study of CIRAP recycling for surface and base together, 14 representative roads are adopted, 219.7 kilometers totally. At the same time, for the study of CIRAP recycling for base only, 9 representative roads are adopted, 47 kilometers totally. For the cement stabilized base course in cold areas, the influence of the low-temperature transverse shrinkage crack should not be neglected, so the modification of PCI is necessary. In the northeast of China, the spacing of this kind of lateral low-temperature shrinkage is 20 meters, which reduces the value of PCI by 15 points. To eliminate the influence of low-temperature transverse shrinkage crack, PCI should be increased by 15 points 4 years later. Calculate the service life of the road when PCI reduces to 60 (the lowest acceptable level). Correction method for PCI is not to modify the original data but the data four years later, because the low-temperature transverse shrinkage crack would be stable after four years since a road was recycled in cold regions.

With this corrected model, the relationship between PCI and the service time is built successfully. For the two different recycling modes, the model equation is obtained respectively. And by means of fitting the trend line, a preliminary forecast life of recycling asphalt pavement is established, results shown in Table 1.

Table 1. Preliminary Forecast of the Service Life for CIRAP

Regeneration Way	Model Equation	Service Life (years)
Recycling in Surface and Base Course	y= -4.76x+100	8.41
Recycling in Base Course	y= -4.73x+100	8.45

The data in the table kept three significant figures.

SERVICE LIFE PREDICTION METHODS FOR RAP

First method. According to the information of the CIRAP, such as the rebound deflection before and after recycling, the variation of the PCI during the service time after recycling, traffic flow and traffic levels, combined with the pavement performance prediction model, the statistics of the existing recycling pavement service life is made as a kind of semi-theoretical and semi-experimental method which can look up to the table. The results include different traffic levels, different conditions of rebound deflections and different cold in-place reclaimed methods.

In accordance with the light, medium, heavy class traffic levels and the rebound deflection before regeneration, a number of levels are divided. The average value on the basis of classification is calculated, as the service lives under different conditions. For the temporary blanks, more accumulation would contribute to the accomplishment in the future. At present, the results of the service life predicted by this method are shown in Table 2 and Table 3. ("-") represents lack of data in the tables)

Table 2. Forecast Life of CIRAP (Recycling for Surface and Base together, years)

Traffic Level Rebound Deflection before Regeneration (0.01mm)	Light	Medium	Heavy
60-90	10.7	8.7	-
90-120	-	6.7	5.6
120-150	-	4.6	-
150-180	-	4.4	-

Table 3. Forecast Life of CIRAP (Recycling for Base only, years)

Traffic Level Rebound Deflection before Regeneration (0.01mm)	Light	Medium	Heavy
40-60	-	6.4	4.8
60-80	-	-	4.6
80-100	7.3	-	4.7

Second method. The second forecast pavement life model is an equation that relates service life of recycling pavement to a combination of some extrinsic ‘traffic factors’ (traffic volume, average annual growth rate of traffic volume) and intrinsic factors (design deflection, actual rebound deflection, etc.). According to serviceability limit states ($I_d = I_0$), adopting the equation (3.17) and (8.0.5) of The People’s Republic of Chinese Asphalt Pavement Design Specification (APDS), by the indefinite coefficient algorithm and the back calculation method, the second forecast method is put forward.

Chinese Asphalt Pavement Design Specification (3.17) and (8.0.5) as follows:

$$N_e = \frac{[(1+\gamma)^t - 1] \times 365}{\gamma} N_1 \eta \quad (3.1.7)$$

$$l_d = 600 N_e^{-0.2} A_c A_s A_b \quad (8.0.5)$$

where N_e : the cumulative equivalent axle of a roadway's design life; N_1 : the equivalent axle load action frequency in the designed traffic lane in the beginning operation period of roads; γ : the average annual growth rate of traffic volume; η : lane coefficient (=1.0 for a single lane, 0.6~0.7 for a dual lane); l_d : RSRDV (0.01 mm) short for the road surface rebound deflection value; A_c : the road rating coefficient (= 1.0 in high-grade highway); A_s : the coefficient of the surface type (= 1.0 in asphalt concrete surface layer); and A_b : the coefficient of the subgrade base type.

According to (8.0.5), setting the 600 to indefinite coefficient as h , to calculate the road surface rebound deflection value after regeneration as l_s . (1.1) as follow:

$$l_s = h N_e^{-0.2} A_c A_s A_b \quad (1.1)$$

Substituted l_s into (3.1.7), applying the back calculation method, to get (1.2):

$$T = \log_{(1+\gamma)} \left[1 + \frac{\gamma}{365 N_1 \eta} \left(\frac{l_s}{h A_c A_s A_b} \right)^{-5} \right] \quad (1.2)$$

To figure out the value of h , firstly using (3.1.7) to get the value of N_e , secondly substituting $l_0 = l_s$ (serviceability limit states) into (1.1).

For example: selecting 8 roads of recycling pavement in base course of Hei-Da region and He-Da regions, the assurance rate was 95%, to calculate the representative value of h .

$$h = \bar{h} - Z_a S$$

where $\bar{h}$: the average value of h (8 roads); S : standard deviation of h ; Z_a : reliability coefficient (=1.645, when assurance rate = 95%).

In the end, the representative value of h was 434.13, assigning it as 430. Substitute h into (1.2), to get (1.3) as an integrated prediction method for the service life:

$$T = \log_{(1+\gamma)} \left[1 + \frac{\gamma}{365 N_1 \eta} \left(\frac{l_s}{430 A_c A_s A_b} \right)^{-5} \right] \quad (1.3)$$

where T : the service life of the pavement recycling for base course only (years); N_1 : the equivalent axle load action frequency in the designed traffic lane in the beginning operation period of roads after recycling; γ : the average annual growth rate of traffic volume after recycling; l_s : the road surface rebound deflection value after recycling (0.01 mm); the meaning of η , A_c , A_s , A_b equals to (3.17) and (8.0.5).

CONCLUSIONS

This paper studied the service condition of CIRAP in cold regions of China, and the results are concluded below.

(1) The technology of CIR can effectively improve the strength of the old asphalt pavement. Based on the improvement of SSI within recycling, the capacity of this technology was analyzed to enhance the structural strength. After recycling, the SSI increment followed the normal distribution law. The expectation is in the vicinity of 0.7 for CIRAP recycling in surface and base course together, and is 0.6 for CIRAP

recycling in the base course only. From comparison, the ability to enhance structural strength would be better for CIRAP recycling in surface and base course together.

(2) The established pavement performance prediction model can be used to predict the PCI of CIRAP. According to the test data of recycling roads, the pavement condition index was analyzed. Based on the data structure, the model function was established. The service lives for two kinds of recycling roads were assessed. It was found that the average life of recycling pavement in surface-base course was 8.41 years, and the recycling pavement in base course was 8.45 years.

(3) Two methods to predict life of CIRAP were developed for comparison. The first method required larger amounts of data than the second one when established, while the second method relies on the forecasting formula. Furthermore, a larger amount of the data would improve the credibility of the results.

The whole research for the actual service condition of CIRAP depends on 14 representative roads, which are approximately 450 kilometers long. The establishment of the model was significantly controlled by the amount of data. The precision needs to be further improved. Based on the method of this research, further modification of the model can be achieved with more data.

ACKNOWLEDGMENT

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Comparative Study on Performance of Recycled Asphalt Mixtures with Ordinary and High-Viscosity Rejuvenating Agent

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ABSTRACT

This paper presents a research study to evaluate performance of recycled asphalt pavement (RAP) mixtures with ordinary and high-viscosity rejuvenating agent. Asphalt content in RAP, properties of recycled asphalt, and aggregate gradation curve were determined through centrifugal extraction test. Rutting test, freeze-thaw splitting and bending test were utilized to compare the impact on the percentage of RAP and rejuvenating content. The results indicated that with the increase of the RAP content, high temperature stability will increase, whereas the low-temperature deformation capacity and water stability of recycled mixture will decrease at a certain degree. Compared to RA-1 with higher viscosity, RA-75 can also help improving the recycled mixture in high temperature performance. But the improvement is not significant in recycled mixture with low percentage of RAP content.

INTRODUCTION

Reclaimed asphalt pavement (RAP) is combined with new aggregate, new asphalt, or recycling agent to produce hot mix asphalt (HMA) through milling, recycling, crushing, and screening for excellent road performance especially under conditions of increasing traffic. Since the old materials are recycled in the construction process, the road geometrics and thickness can be well maintained. The earliest studies extensively conducted by American forcing on rejuvenating agent development, recycling mixture redesign, construction equipment and other aspects were becoming increasingly in depth. For example, Chaffin J.M. from Colorado analyzed the relationship between compositions of the rejuvenating agent with its physical properties and aging characteristics (Chaffin 1996). Malpass from North Carolina State University proved that reclaimed asphalt mixture can do better in road performance than ordinary hot mix asphalt based on tests on 4 kinds of RAP designed by different aggregate gradation and rejuvenating agent with several rheological properties and dosages (Malpass 2003). Vistintine evaluated rheological properties and rutting resistance by treating RAP at the content level of 15%, 30% and 40% to get the economically optimal dosage of RAP in life cycle (Vistintine 2011). In recent years, more and more comprehensive and systematic studies on recycled asphalt mixture have been completed by domestic research institutes and related technical personnel due to the fast development of rejuvenating agent. Experimental analysis

carried out on the reclaimed asphalt pavement by Zhuang dongliang (Zhuang and Xu 2010), Shen benxian (Wang and Shen 2011), and Ye qin (Ye 2008) demonstrated the road performance of RAP. Compared to conventional mode of asphalt pavement maintenance, the use of higher percentages of reclaimed asphalt pavement decreases the environmental impacts and cuts down considerable cost of asphalt binder and quality aggregates at the same time.

EXPERIMENTAL PROGRAM

Materials. In this study, RAPs have been recycled from the milled materials of HaA highway, Erhuan highway and Husui highway (abbreviated as RAP from HA, EH, HS(C) and HS(F)). The base 90# asphalt and limestone aggregates(sieved to 0-3mm, 3-5mm, 5-10mm, 10-20mm and the HS RAP was sieved with 9.5mm previously) produced for the asphalt mixture were supplied from Panjin in Liaoning Province.

Table 1. Properties of Aged asphalt and the reference binder

Asphalt Sources	Penetration (0.1mm)	Softening point (°C)	Ductility (cm,15°C)
HA	12.4	72.9	19.4
EH	51.3	62.5	19.3
HS(F)	56.6	55.3	32.9
HS(C)	47.8	56.5	30
90#	80~100	≤45	≤100

Table 2. Asphalt Content in RAPs

Asphalt Sources	HA	EH	HS
Asphalt Content (%)	4.2	3.3	5.3

Table 3. Properties of RA-1

Test Items	Specification	Specification Limits	Results
60°C Viscosity (sc)	T0619	50--175	91.58
Flash point(°C)	T0633	≥220	226
Saturated Phenol Content (%)	T0618	≤30	25.87
Aromatic Phenol Content (%)	T0618	Inspection Records	33.46
Viscosity Ratio before and after RTFOT	T0619	≤3	1.11
Mass Ratio before and after RTFOT	T0609	≤4, ≥-4	3.52
	T0610	≤4, ≥-4	3.78
15°C Density (g/cm3)	T0603	Inspection Records	0.997

In order to characterize the properties of the RAP, Abson asphalt recovery test (ASTM D 1856), sieve analysis, penetration test, ductility test and softening point test (JTG F41-2008) were performed. Table 1 presents the properties of the recycled asphalt from RAPs. The asphalt contents in the RAP are presented in Table 2.

According to the rejuvenating capability, American Society for Testing

Materials (ASTM) D 4552 classified rejuvenating agent as RA-1, RA-5, RA-25, RA-75, RA-250, and RA-500. Two rejuvenating agents (RA) were considered to be utilized for this study. Table 3 and Table 4 present the properties of RA-1 and RA-75.

Table 4. Properties of RA-75

Test Items	Specification	Specification Limits	Results
60°C Viscosity (sc)	T0619	4501-12500	10097
Flash point (°C)	T0633	≥220	185
Saturated Phenol Content (%)	T0618	≤30	35.78
Aromatic Phenol Content (%)	T0618	Inspection Records	38.25
Viscosity Ratio before and after RTFOT	T0619	≤3	1.04
Mass Ratio before and after RTFOT	T0609	≤3, ≥-3	2.71
	T0610	≤3, ≥-3	2.82
15°C Density (g/cm ³)	T0603	Inspection Records	0.993

Production methods. Since the differences in rejuvenating capability of RA and the grade of aging recycled asphalt, multi-factor should be considered in the amount of rejuvenating agents to obtain the desired intermediate binder with equivalent penetration to that of the reference binder (base 90# asphalt). Therefore, three parameters (penetration at 25 °C, ductility test and R&B softening point) were chosen to be characterized the replicates of each binder compared with those of reference binder. For different degrees of aging asphalt, the rejuvenator agents restore their performance unchangeably. And then the guidelines for virgin asphalt selection and evaluation of aged asphalt have been proposed by “Superpave mix design method in old asphalt pavement recycling in the Technical Manual” showed a good linear relationship between the bitumen viscosity and penetration in double logarithmic confirmed by relevant researches.

$$\lg \lg \eta_{mix} = (1 - \alpha) \lg \lg \eta_{old} + \alpha \lg \lg \eta_{new} \quad (1)$$

Where,

η_{mix} = 60 °C viscosity of final binder after mixing (Pa·s),

η_{old} = 60 °C viscosity of aged binder before mixing (Pa·s),

η_{new} = 60 °C viscosity of virgin binder before mixing (Pa·s),

α = mass fraction of virgin binder.

In this study, in accordance with “JTG E-20 2011”, production method for the final binders should be assessed by penetration value as grades of asphalt were ranked by penetration classification principle in specification. Therefore, a desirable RA content could be firmly accounted if final binder met the requirement of penetration limits of base 90# asphalt.

Construction mix proportion could be concisely designed owing to

determination of properties of RAP, virgin aggregate and virgin binder (mixed with RA) with reference to specification (JTG F41-2008).

The effect of RAP content on the mechanical properties of warm mix asphalt has been determined by rutting test, freeze-thaw splitting (or immersed Marshall Test) and bending test of small beam. Five contents (30%, 40%, 50%, 60%, and 70%) have been considered to investigate the effect of variation of RAP content on WMA and two contents have been chosen to compare the rejuvenating capability of RA-1 and RA-75. The sample mixing without RA was utilized as the reference.

RESULTS AND DISCUSSIONS

Table 5 and Table 6 depicted the properties of recycled asphalt mixed with multiple percentages of RA-1 and RA-75 in order to determine an optimal RA content for each sorts of RAP.

Table 5. Properties of Aged asphalt mixed with RA-1

Asphalt Sources (mixed with RA-1)	Penetration (0.1mm)	Softening point (°C)	Ductility (cm,15°C)
HA(20%)	54.6	50.5	>100
HA(23%)	76.1	48.1	>100
HA(25%)	110.6	45.0	>100
EH(12%)	92.3	52.4	38.3
HS(10%)	123.8	45.4	53.2
HS(8%)	105.7	47.1	44.6
HS(6%)	87.6	49.7	36.9
Base 90# asphalt	80-100	≥42	>100

As seen in Table 5 and Table 6, it's no doubt to see that with the addition of rejuvenating agent, penetration value and ductility went up while the softening point declined. This may be due to the role of rejuvenator that recovered the missing chemical composition and its rheological properties.

Table 6. Properties of Aged asphalt mixed with RA-75

Asphalt Sources (mixed with RA-75)	Penetration (0.1mm)	Softening point (°C)	Ductility (cm,15°C)
HS(35%)	158.4	—	99.8
HS(30%)	157.0	—	—
HS(15%)	81.2	48.1	80.9
HS(6%)	41.6	59.0	33.1
HS(3%)	57.8	—	—
Base 90# asphalt	80-100	≥42	>100

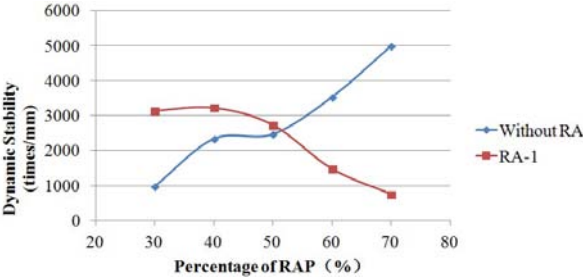
Table 7. Optimal Contents of Rejuvenating Agents

Asphalt Sources	HA	EH	HS(RA-1)	HS(RA-75)
RA contents(%)	24	12	6	22

Table 8. Characteristics of WMA mixed with HS RAP rejuvenated by without RA, RA-1 and RA-75

	RAP contents(%)	Dynamic Stability(time/mm)	Frozen-thaw and Split Residual Strength Ratio(%)	Flexural Strain (μϵ)
Without RA	30	967	91.6	2534
	40	2332	90.9	1952
	50	2466	94.4	1932
	60	3523	93.8	1718
	70	4981	91.5	1643
RA-1	30	3150	89.4	2713
	40	3233	81.4	2804
	50	2742	89.0	2496
	60	1483	88.6	2392
	70	754	88.7	2480
RA-75	30	4200	83.9	2773
	50	4846	90.3	3025

The optimum RA contents to be added into mixture for RA-1 and RA-75 with respect to the values are given in Table 7, and the aged asphalt sample designated as HS was selected to inspect the superiority of mechanical properties of WMA rejuvenated by RA-75 over that rejuvenated by RA-1 as the HS-RAP was less-aging enough to be suitable for both kinds of RA samples. Rutting test, freeze-thaw splitting(or immersed Marshall test) and bending test of small beam were performed by specific contents of RAP listed above in order to investigate the trends varied with percentage of RAP and class of RA. Table 8 presents results of mechanical tests conducted on HS RAP mixed with Without RA, RA -1 and RA-75.



a)

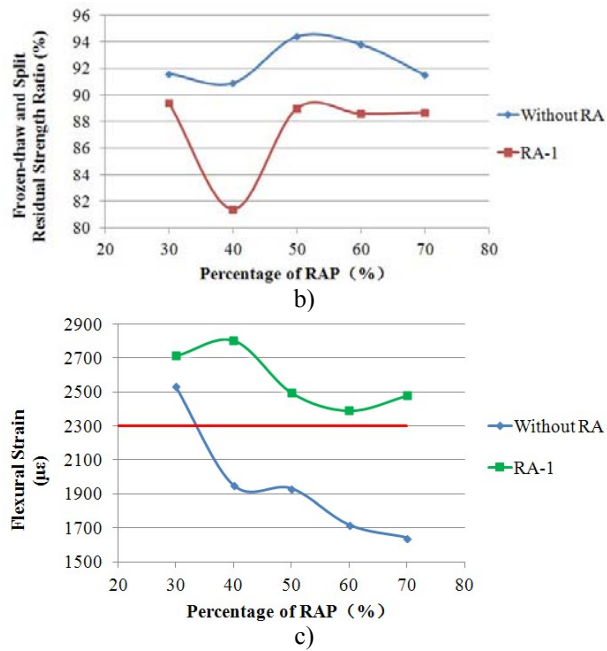
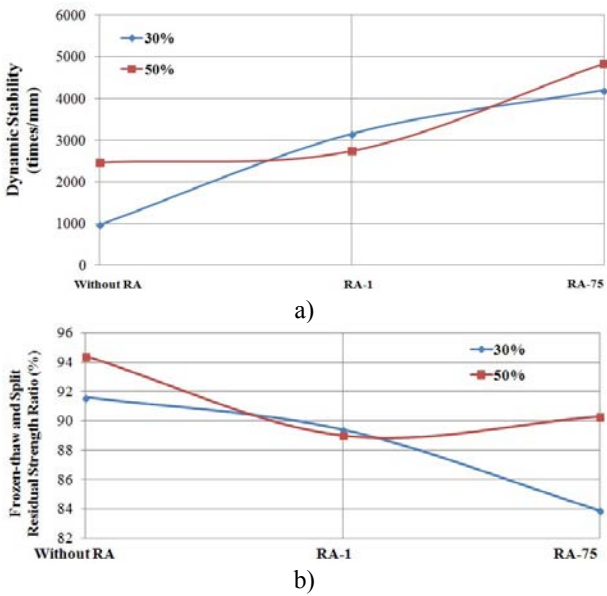


Figure 1. Results of mechanical tests for WMA varied with RAPs(ranging from 30% to 50%) and RA(without RA and RA1).



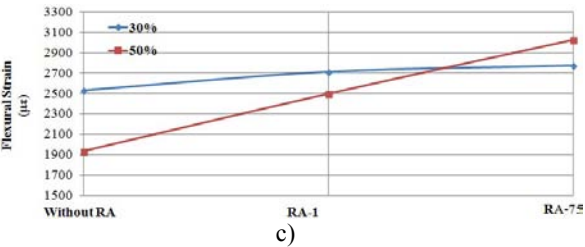


Figure 2. Results of mechanical tests for WMA varied with RAPs(30% and 50%) and RA(without RA, RA-1 and RA75).

As illustrated as Fig. 1 and Fig. 2, high temperature stability goes up with increase in the percentage of RAP without RA which has been achieved in relevant study. With the rising of percentages of RAP, a peak of Dynamic Stability value appears at about 30% or 40% of RAP mixed following by a steep decline after 40% of RAP and this phenomena has been verified in all of 3 RAP samples. RA-75 is provided with a better rejuvenating capability in improving dynamic stability of WMA than the RA-1 as the higher viscosity of former. Therefore, the results make RA-75 suitable for the less-aged RAP to obtain an equal high temperature performance to that of modified asphalts.

Raps generally slightly affect moisture susceptibility which can suffice the limits of specification in most of engineering projects under the circumstance of proper quality control. However, no significant improvement has been confirmed by RA75 in terms of moisture susceptibility and it's reasonable to be replaced by RA-1 economical requirements.

No significant variation is observed on the value of flexural strain when the content of RAP below 40% while it is still necessary to recover the low temperature crack resistant by RA-75 when high percentage of RAP is determined. And it can also be substituted by RA-1 owing to its negligible capability in rejuvenating low temperature performance at a low percentage of RAP.

CONCLUSIONS AND RECOMMENDATIONS

It was shown that reclaimed asphalt pavements, suffering from the long-term influence from nature such as oxygen, ultraviolet radiation, and temperature variations have changes in chemical composition and rheological properties expressed as the increase of softening point with the reduction of penetration value and ductility.

It can also been found that linear relationship between the bitumen viscosity and penetration in double logarithmic cannot be confirmed in this study. However, the rejuvenating capability has the similar performance when the RA is mixed with aged asphalts which can recover the properties to obtain the limits of specifications in force.

Compared with RA-1, RA-75 showed better rejuvenating capability in improving high temperature stability and low temperature crack resistant with a high percentage of RAP(above 40%) while both of them are unable to significantly promote improvement of moisture susceptibility.

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Measurement Method of Blending Status between Virgin and Aged Binder in Recycled Asphalt Mixtures – A Literature Review

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ABSTRACT

Design procedure of hot in-plant recycled asphalt mixture assumes that the residue aged binder in reclaimed asphalt pavement (RAP) is fully blended with virgin binder. However, recent researches show that the interaction between aged binder and virgin binder may not achieve a total blending status. The difference between design assumption and actual blending status means that insufficient asphalt binder used in mixtures may cause dry hot mix asphalt (HMA) and premature pavement distress. Therefore, it is significant to evaluate the reasonableness of design assumption and mixing quality of recycled asphalt mixtures to measure the blending status between aged binder and virgin binder. In this paper, four typical measurement method of blending status, binder-marked method, difference-identified methods, staged extraction method, and indirect performance measured method, are summarized. The future development in measurement method is also discussed.

INTRODUCTION

Hot in-plant recycling HMA is one of commonly used recycling pavement technology. Virgin binder, virgin aggregate and RAP are mixed as recycling hot mix asphalt based on optimal asphalt binder content, aging degree of binder and aggregate gradation of reclaimed asphalt pavement (RAP). Hot in-plant recycling has been favored in the light of the increasing cost of asphalt, the scarcity of quality aggregates, and the pressuring need to preserve the environment.

According to the difference in level of interaction between aged binder in RAP and virgin binder, three possible blending statuses are identified: no blending (black rock), total blending and partial blending (Figure 1) (*Al-Qadi et al. 2007*). Current design procedure of recycling HMA assumes that the aged binder in RAP is fully blended with virgin binder. The common design procedure is conducted as follow. Firstly, aged binder in RAP is extracted. Secondly, virgin binder with different content is fully blended with extracted aged binder and the performance of mixed binder is tested. Finally, the virgin binder content which meets all the performance criteria is determined as optimal content.

Recent research found that virgin binder can totally blended with aged binder when RAP content is lower than 30%. However, it is possible that only partial aged binder around old aggregate blended with virgin binder when RAP content larger than

30% (McDaniel *et al.* 2000, West *et al.* 2009, McDaniel *et al.* 2012). It means that the actual total binder content (virgin binder and partial aged binder) of recycled asphalt mixture is lower than optimal binder content. The actual mixed binder will present lower penetration index and higher softening point compared to design mixed binder with total blending status. Greater differences in pavement performance were also found between actual mixtures and design mixtures (Al-Qadi *et al.* 2009, Aurangzeb *et al.* 2012).

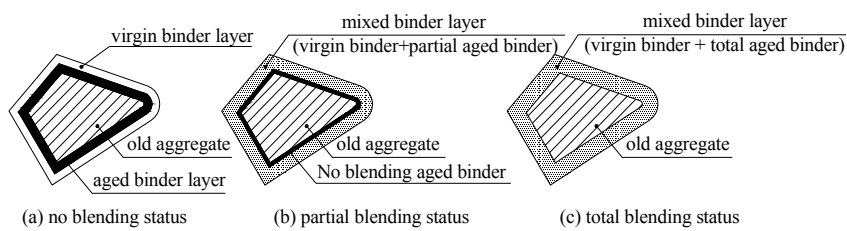


Figure 1. Three statuses between virgin binder and aged binder

In addition to the difference of blending status between actual mixing condition and design assumption under some special conditions, some researchers also found that the blending process between virgin binder and aged binder would continue until it reached total blending status after the recycling HMA was mixed and paved (Figure 2) (Carpenter *et al.* 1980, Nouredin *et al.* 1987, Kadar 1996). Continuous changes of blending status would lead to continuous changes in pavement performance of recycling HMA. Carpenter measured the mechanical properties of the recycling HMA and found that the $|E^*|$ and rutting resistance performance of mixture specimens decreased significantly two weeks later compared to that of these specimens when they just compacted (Carpenter *et al.* 1980). The findings were verified by Nouredin (Nouredin *et al.* 1987) and Kadar (Kadar 1996). Kadar also considered that the blending process between virgin binder and aged binder would continue 3-6 months after the recycling HMA were mixed and paved. Pavement performance of mixtures, which meet the design criteria when mixture were just paved, would drop to below the design criteria several months later and some premature pavement distresses would emerge (Figure 2).

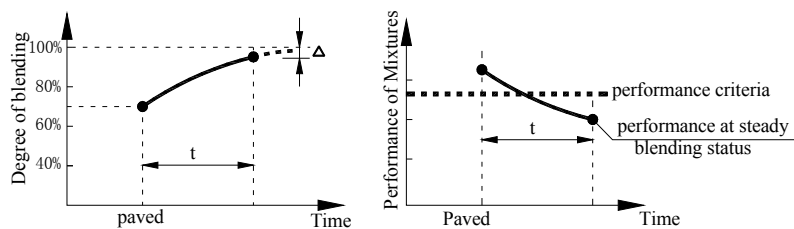


Figure 2. Change of blending status and its influence on mixture performance

Therefore, it is significant for evaluating the design and construction quality of recycling HMA to measure the blending process between aged binder and virgin binder

and its effect on mixture performance. Many researchers applied some methods to measure the blending status between aged binder and virgin binder in recycling HMA. In this paper, these methods were summarized and analyzed to provide reference for the study on related research area.

MEASUREMENT METHOD OF BLENDING STATUS BETWEEN AGED BINDER AND VIRGIN BINDER

Typical measurement method of blending status concluded binder-marked method, difference-identified methods, staged extraction method and indirect pavement performance measured method.

Binder-Marked Method

Marking of binder means that the molecules are manipulated in such a way, that they can be detected and recognized. This manipulation could either be done by altering the atoms (for example use of neutron radiation to produce C13 or D isotopes) or slightly changing the chemical composition. However, the changing of atom structure of asphalt binder may lead to an unsatisfactory result that the testing data does not reflect the real blending properties of original asphalt binder. Therefore, the latter method is more commonly used.

In 2003, Karlsson and Isacsson used eight substances, such as Glycerol, diphenyl silane, polystyrene etc, as markers to label binder (*Karlsson and Isacsson 2003*). Fourier Transform Infrared Spectroscopy with Attenuated Total Reflectance (FTIR-ATR) was applied to investigate the influence of temperature, viscosity of the diffusion medium, and diffusant size on diffusion characteristics. Many other observation apparatus are also used to measure the diffusion process between virgin binder and old binder. In 1983, Lee used Energy Dispersive X-ray Spectroscopy (EDS) to measure the blending process between aged binder and virgin binder marked by Titanium (*Lee et al. 1983*). The results present that there exists a clear interlayer between two kinds of binder.

Most of above researches only focused on diffusion characteristics between two binders, without considering the influence of mixture structure. Therefore, the results couldn't reflect the real blending properties between virgin binder and aged binder in real recycling HMA structure. Some researchers used image technology to observe the blending process between two binders in mixture specimens directly. Nguyen colored virgin binder by adding pigments (iron oxide particles) and observed the slice of mixture specimens using image technology (*Nguyen 2009*). The homogeneity of recycling HMA could be visual assessed. A microscopic observation technique was proposed by Navaro to the way the virgin binder and the reclaimed binder blend together (*Navaro et al. 2012*). Aged binder, virgin binder, new and old aggregates in images could be distinguished using grey level. Two grey-related indicators were proposed to characterize the blending properties of binders.

According to the above studies, the selecting of marker is the key for binder-marked method. The marker shouldn't change the diffusion property of marked binder, otherwise, the reliability of measured results will be greatly reduced.

Difference-Identified Methods

Difference-identified methods try to measure the blending process by capturing the identifiable differences between binders (*Zhang 2011*). Compared with binder marked method, it observed the blending process between aged binder and new binder directly without altering atoms structure or chemical composition of binder.

Some special rejuvenator, which could easily observed under the ultraviolet rays irradiation, was selected and mixed with RAP (*Davidson et al. 1977*). The diffusion and blending properties of rejuvenator could be measured by observing the change of its distribution status in mixture specimens. Research results found that an inverse proportion relationship was shown between the rejuvenator content and the time of rejuvenator diffuse to the whole specimen. However, it is difficult to popularize for this method as only some special rejuvenators could be observed directly in this method.

Yang et al. mixed virgin binder, new aggregate with RAP containing mineral powder (*Yang et al. 2011*). The proportion of mineral powder in asphalt mortar around new aggregate accounts for total mineral powder was used to characterize blending status between virgin binder and aged binder indirectly. The influences of heating temperature of RAP with aged binder, mixing time of recycling HMA are also analyzed. In this study, mineral powder is determined as the identifiable differences between binders. However, only 9.5mm particles of new aggregate and 4.75 mm sieve passing particles of RAP were mixed with virgin binder in this method. The adding content of virgin binder would also be equal to the content of aged binder in RAP. These limitation make this method can't reflect the real composition of recycling HMA, which greatly reduce the applicability of this method.

Although difference-identified methods don't alter atoms structure or chemical composition of binder, it is very difficult to find easily identified differences between binders. Therefore, this method isn't widely used.

Staged Extraction Method

In this method, staged extraction is used to obtain asphalt binders from different layers coating the RAP aggregates (Figure 3). The performance differences among binders of different layers can represent the blending status between virgin binder and aged binder indirectly.

In 1979, Zearley and Carpenter adopted two stage and four stage extraction method respectively to measure the blending process (*Zearley 1979*). The results indicated that the blending process between virgin binder and aged binder would continue for 3-6 months after the recycling HMA was paved. In 2005, Huang used four stage method to test the difference of viscosity and $|E^*|$ among different layer's binders (*Huang et al. 2005*). The results found that the inner layer of binder coating around RAP aggregates is stiffer than the outer layer of binder. It indicates that the blending status between virgin binder and aged binder showed non-homogeneity (partial blending status).

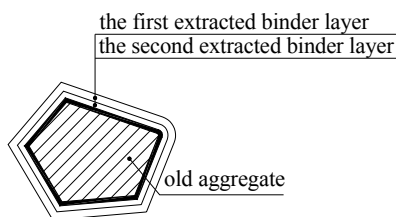


Figure 3. Staged extraction method

Staged extraction method roughly classified the binder around the RAP aggregates as several layers. The rules of classification depend on subjective experience of researcher. There have no distinct boundary between each binder layer. Therefore, this method only qualitatively analyzed the blending status between binders at one moment.

Indirect Performance Measured Method

Indirect pavement performance measured method is a kind of indirect measurement method by comparing the change of mechanical and pavement performance of recycling HMA during the blending process between binders. Bonaquist et al. predicted the $|E^*|$ of recycling HMA at total blending status using Hirsch model based on the volume properties of mixtures and stiffness of extracted recycled binder (Bonaquist 2007, Christensen et al. 2003). The blending status could be determined by comparing the predictive value and measured value of $|E^*|$. In this method, Witczak model could also be used to predict the $|E^*|$ of recycling HMA (Copeland et al. 2010, Andrei et al. 1999). In 2012, McDaniel and Shah adopted Bonaquist's method to analyze the blending degree between binders of recycling HMA with different RAP blending rate (McDaniel et al. 2012). The results indicated that a total blending status was found between binders when RAP content was lower than 15%. However, it presented partial blending status when RAP content was higher than 40%. McDaniel and Stephens analyzed the influence of RAP content and preheating time of RAP on blending status by testing the changes of indirect tensile strength and unconfined compressive strength of recycling HMA (McDaniel et al. 2000, Stephens et al. 2010).

Indirect performance measured methods characterize the blending degree between binders by comparing the difference of mixture performance under total blending status and real blending status. However, the performance of mixture is affected by too many factors, e.g. aggregate's properties, the change of mixture volume properties, mix temperature and mix time etc. The change of performance is not necessary caused by the change of blending status. The accuracy of prediction model is also questionable. Therefore, the measured result of this method has greater variability.

DISCUSSION

In summary, binder-marked method and difference-identified methods belongs to microscopic method and staged extraction method and indirect performance measured method belongs to macroscopic method. Microscopic method try to measure the diffusion properties between virgin binder and aged binder directly using some microscopic test instruments e.g. scanning electron microscope (SEM), FTIR, EDS, GEL Permeation chromatography. Because of the limitation of testing technology based on these microscopic instruments, it is difficult to measure mixture specimens directly. Therefore, most of researches only focus on diffusion properties between binders. These research results related to diffusion properties can't characterize the blending process between virgin binder and aged binder in real recycling mixture structure. Although some researcher try to measure the mixture specimens directly, it is still very difficult to quantitative analyze the blending status.

With regard to microscopic method, another issue is that the testing results may not represent the real blending properties of marked binder using some markers e.g. pigment, photoactive substance. In order to facilitate observing, it assumes that the diffusion properties of marker are the same with that of marked binder in microscopic method. In fact, the blending of binders is a kind of blending among components of binders. Composition of binder structure is very complex and the diffusion properties of different components are also differed from each other. Therefore, the measured diffusion properties of marker may not reflect the actual diffusion properties of marked binder.

Because of disadvantages of microscopic measurement methods, some researchers measure the blending properties between binders by observing the change of recycling HMA macroscopic performance. As far as stage extraction method is concerned, there are still some disadvantages. Firstly, it is a kind of qualitative analysis method by comparing the performance of different layer's binder around RAP aggregate particle as the classification of binder layers depends heavily on the experience of researchers. Secondly, as discussed above, the measurement result of this method has greater variability. The thickness of binder layer around different size of particles may be different. Thirdly, the solubility of different binder components in solvent, e.g. trichloroethylene, is different. It is possible that the more contents of light binder component are extracted out in the first few extraction stages and the more contents of heavy binder component are extracted out in the last few extraction stages. Obviously, the extracted binder with more heavy binder component show more aging properties than the binder with more light binder component. The results may enable us to misunderstand the real blending status between binders. Lastly, staged extraction method is a kind of destructive test. The recycling mixture specimens will be destructed after its binder is extracted out. Therefore, we can't measure the continue change of blending status between binders.

As for indirect performance measured method, currently, the most commonly used method is the method proposed by Bonaquist (*Bonaquist 2007, Christensen et al. 2003*). However, it is also a kind of destructive test. This method can't quantitatively explain the relationship between blending process and the performance change of recycling mixture from perspective of mechanism.

In conclusion, all four commonly used methods have the pros and cons. Therefore, with the development of microscopic test technology, it is significant to propose a

method of continuously measuring the diffusion properties of virgin and aged binder around the RAP aggregate particles based on the real recycling mixture structure. At the same time, it is also a big challenge to build the model between blending process of virgin and aged binders and the performance change of recycling mixture.

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Study on the Performances of Direct Coal Liquefaction Residue Modified Mixture Asphalt

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ABSTRACT

The properties of two different asphalt mixtures blended with DCLR (direct coal liquefaction residue) and binder using two processing methods were studied in this paper. One processing method is to blend powdered DCLR and binder, and the other processing method is to blend molten DCLR and binder. The optimum content of DCLR was determined using PG system and furfural extract oil was used to improve the low-temperature properties of the blends. The performance of DCLR modified asphalt mixture with the two different blends were also studied in this paper. At first, the DCLR-13 was designed using the Marshall method. Then, the rutting test, the freeze-thaw splitting test, and the low-temperature bending test were conducted on the DCLR-13. Finally, the performances of DCLR-13 with the two different blends were evaluated and compared. The test results indicated that the performance of DCLR modified asphalt mixtures satisfied the specification. The processing method has an influence on the performance of DCLR modified asphalt mixtures.

INTRODUCTION

China is a country with rich coal and gas but little petroleum. At present, the explored coal reserves reach 1.3983 trillion tons, ranking the second place in the world. However, petroleum resources are relatively rare in China. In 2012, the gap of supply and demand for petroleum in China is nearly 300 million tons which caused China's dependence on imported petroleum climbs up to 60%. In order to relieve the shortage of petroleum resources in China, it has been an important energy strategy trend to produce petroleum from coal. Our demand for liquid fuel can be basically met by means of coal liquefaction. The coal liquefaction can be classified by direct liquefaction and indirect liquefaction according to the processing method, among which the direct coal liquefaction for petroleum brings in a higher conversion rate and lower cost. Therefore, it witnessed a quick development.

In the direct coal liquefaction process, different from liquid fuels such as gasoline and diesel, the main byproduct liquefied residues are also produced which accounts for 30% of total feed coal. In fact, the direct coal liquefaction residue (DCLR) includes 30~50% of heavy oil and binderene, which is similar with Trinidad lake binder (TLA, a natural modifier to improve the binder properties) in composition and some physical properties. Among non-polymer additives, with a good modification property, TLA has been widely applied all over the world. Many important projects in China also use TLA, e.g. Jiang-Yin Bride, western runway of Beijing Capital International Airport and Zi-Yang Section of Chengdu-Chongqing Expressway. With the construction of expressways in China and improvement of common highway, the demand for TLA also increases. However, the application of TLA is restricted since the TLA source is limited and the import price is high.

DCLR is a substance most similar with TLA in properties. Hence, the study on new technology that the blends with DCLR and binder are secondarily developed to a binder modifier which is used to replace the TLA to improve the binder properties has very important scientific significance and application value. On the one hand, DCLR, as a binder modifier, can significantly improve the binder properties, reduce the price of modified binder (DCLR price is only equivalent to a quarter of TLA) and promote the development of binder industry of China. On the other hand, the use of binder modified by DCLR is of positive significance in environmental protection, reasonable use of limited resources, and increase of DCLR utilization.

PROPERTIES OF DCLR

DCLR used in this paper is from the direct coal liquefaction residue manufactured by Mongolia Branch of China Shenhua Coal To Liquid And Chemical Co., Ltd. and its properties are as shown in Table 1.

Table 1. Some Properties of DCLR

Items	Density(g/cm ³)	Softening point(°C)	25°C penetration(0.1mm)
Test results	1.23	170	2

PROCESSING METHOD FOR BLENDING DCLR AND BINDER

DCLR used in this paper is powder and blends with binder using two different processing methods. The processing method I is used as follows: firstly heat the binder, and then add different amounts of DCLR into melting binder directly. In order to ensure that DCLR is fully blended with binder, the blends composed of DCLR and binder is sheared with a shear apparatus for 1 hour at low speed under the temperature of 160°C. The processing method II are as follows: heat the binder and DCLR respectively to melting status, and then blend molten DCLR and binder in a certain proportion. In order to ensure that DCLR is fully blended with binder, the

blends composed of DCLR and binder is sheared with a shear apparatus for 1 hour at low speed under the temperature of 160°C.

DETERMINE THE DCLR CONTENT

The Pen 60-80 binder was used in this paper. At first, the DCLR content was 5%, 10%, 15%, and 20%. Secondly, the different content DCLR and pen 60-80 binder were blended using the processing method I and II. Finally, the properties of blends were tested, including the 25°C penetration, the softening point, the 10°C ductility.

From the test results, it was found that:

1) Whatever the processing method I or processing method II, the 25°C Penetration of the blends were decreased as the DCLR content increased. When the DCLR content was more than 15%, the 25°C penetration of the blends was lower than 40(0.1mm).

2) Whatever the processing method I or processing method II, the softening point of the blends was increased as the DCLR content increased. When the DCLR content was higher than 15%, the softening points of the blends were raised up very slowly.

3) Whatever the processing method I or processing method II, the 10°C ductility of the blends were decreased as the DCLR content increased. When the DCLR content was higher than 15%, the 10°C ductilities of the blends were almost zero.

After the DCLR blended with binder, the low-temperature properties of the blends was sharp degraded, some furfural extract oils was added into the blends to improve the low-temperature properties of the blends. When the 5% furfural extract oil was added into blends, the 10°C ductilities of the blends were improved and met the specifications .So it suggested that the properties of the blends was the best while the 15% DCLR and 5% furfural extract oil was blended with binder. The test results are seen in Table 2.

Table 2. Some Properties of the Blends (Adding 15%DCL and 5%FEO)

Items	25°C Pen	Softening point	10°C ductility	135°C viscosity	Residue after RTFOT		
					Loss	Residual	10°C
Unit	0.1mm	°C	cm	Pa.s	%	%	cm
Method	38	57.3	3.3	0.49	0.07	61	1.2
Method	28	59	6.2	0.48	0.03	58	1.1

THE PERFORMANCES OF THE MIXTURE ASPHALT

The AC-13 DCLR modified mixture asphalt was designed used the blends as binder. The optimum binder content was 5.4%. The performances of the mixture asphalt were evaluated by using the rutting test, the freeze-thaw splitting test, and the

low-temperature bending test. In order to compare the performances of the DCLR modified mixture asphalt, the control mixture asphalt based on the same gradation was also made. The gradation of AC-13 is seen in Table 3.

Table 3. The Gradation of AC-13

Sieve(mm)	Passing ratio (%)	Limit value (%)
16	100	100
13.2	96.5	90~100
9.5	74	68~85
4.75	57.2	38~68
2.36	45.5	24~50
1.18	29.6	15~38
0.6	20.9	10~28
0.3	13.4	7~20
0.15	9.4	5~15
0.075	6.8	4~8

High-temperature performances

The anti-rutting capacities of the mixture asphalts were conducted by using the rutting test according *the Standard Test Methods of Bitumen and Bitumen Mixture for Highway Engineering (JTG E20-2011)* in China. Table 4 is the test results of the mixture asphalt.

Table 4. The dynamic stability of the mixture asphalts

Processing method	Dynamic stability (pass/mm)		Limit values
	DCLR modified mixture	control mixture	
Processing method I	4182.8	3183.7	>1000
Processing method II	4022.0		

According to *the technical specifications for construction of asphalt pavements (JTG F40—2004)*, the rutting resistance of DCLR modified mixture asphalt met specification under the two processing methods. Compare to the control mixture asphalt, the DCLR modified mixture asphalt’s dynamic stabilities increased 40%. It meant the rutting resistance of mixture asphalt could be improved by adding DCLR.

Low-temperature performances

The low-temperature bending test was conducted on the mixture asphalts. Table 5 is the test results of the mixture asphalt.

Table 5. The failure strains of the mixture asphalts

Processing method	failure strain (µε)		Limit values
	DCLR modified	control mixture	
Processing method I	6500.9	1120	>2000
Processing method II	5802.6		

According to the technical specifications for construction of highway asphalt pavements (JTG F40—2004), the anti-cracking capacities of the DCLR modified mixture asphalt satisfied the specification requirement under the two processing methods. Compare to the control mixture asphalt, the DCLR modified mixture asphalt’s failure strain increased 400%. It meant the anti-cracking capacities of mixture asphalt had been increased after adding DCLR.

Water sensibility

The freeze-thaw splitting test was conducted on the mixture asphalt. Table 6 is the test results of the mixture asphalts.

Table 6. The TSR of the mixture asphalts

Processing method	TSR (%)		Limit values
	DCLR modified	control mixture	
Processing method I	85.4	90.1	>75
Processing method II	91.6		

According to the technical specifications for construction of highway asphalt pavements (JTG F40—2004), the water sensibility of the DCLR modified mixture asphalt meet specification under the two processing methods. Comparably, the water sensibility of the DCLR modified mixture asphalt under the processing method II was lower than that of mixture asphalt using processing method I. It meant the water sensibility of mixture asphalt had been reduced when adding DCLR.

CONCLUSION

When DCLR blends with binder, the high-temperature properties of binder could be improved, but the low-temperature properties of binder could be reduced. In order to improve the blend’s properties, the FEO was added into the blends. The performance of the DCLR modified mixture asphalt satisfied the specification requirements, especially their high and low-temperature characteristics. The test results showed that the performance of asphalt mixtures could be increased by adding

DCLR. The processing method had impacts on the performance of the DCLR modified asphalt mixtures. The processing method II was suggested.

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Laboratory Evaluation on Performance of Large Particle Size OGFC Asphalt Mixtures with Different Air Voids

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ABSTRACT

OGFC (open graded friction course) asphalt mixture is one of the bituminous materials of wearing course for flexible pavement. The permeable mixture allows water to flow through the material and drain out to the sides. The material significantly reduces splash/spray from vehicles and reduces vehicle noise. The Air void and Particle size of OGFC asphalt mixture are important factors which influence the material performance properties and permeability. In this research, the large particle size OGFC-20 asphalt mixtures with three air voids (18%, 21% and 24%) were prepared. The temperature-stability, water-stability, and permeability of OGFC asphalt mixtures were evaluated and compared to OGFC-13 (popular used in China) to ensure better drainage, noise reduction quality, and durability. The results showed that the Marshall and dynamic stability reduced with the increase of air voids. Under the same air voids, the stability of OGFC-20 asphalt mixture is significantly more than that of OGFC-13. Furthermore, the permeability coefficient of OGFC asphalt mixture increased as the increasing of air voids. It showed a good linear correlation between them. To balance the performance and the permeability of OGFC asphalt mixture, OGFC-20 with 21% air voids is recommended for wearing course of flexible pavement.

INTRODUCTION

In recent years, due to the large scale of the city construction, more and more natural ground has been replaced by the impermeable artificial structures. According to statistical results, there are more than 50% impermeable areas in many cities of china. The transportation infrastructures cover more than 25% among of them (Xu 2011). As a result of the higher impermeable ratios, the drainage system of the cities reaches to great burden and insufficiently ability to the continually heavy rain-falls. It is easy to cause the urban waterlogging. Furthermore, retention of the rain water on the ground surface will cause a series of problems it the cities, such as decline of the groundwater level, urban heat island, and the serious waste of water resources. Aiming at the problems, the LID (Low Impact Development) has been put forward to reduce the influence of the city development on the ecological environment in

recently years. The main objective of the LID applied in the rain water management and control is rebalancing the city water cycles (Eddy 2008).

OGFC asphalt mixture is one of bituminous materials of wearing course for flexible pavement. The permeable mixture allows water to flow through the material and drain out to the sides. The OGFC, known as porous pavement, contains coarse aggregate more than 70% and 18%-25% air void (USEPA). Due to the rain, water can be temporarily stored in the air voids. Peak rate of runoff will be reduced significantly.

In this research, two gradation types (OGFC-13 and OGFC-20) asphalt mixtures specimens with three air voids (18%, 21% and 24%) were prepared. The temperature-stability, water-stability, and permeability of OGFC asphalt mixture were evaluated in the laboratory to ensure fine drainage, noise reduction quality, and durability.

MIXTURE DESIGN

Raw materials. In this research, 4.75mm and above coarse aggregates were selected as the basalt aggregate. The fine aggregates are limestone. The density of materials has been measured and the results are shown in Table 1. Asphalt with high viscosity asphalt is shown in Table 2 with its main performance index.

Table 1. Mineral aggregate density

Aggregate size (mm)	Apparent relative density (g/cm ³)	The relative density of bulk volume (g/cm ³)	The relative density of dry (g/cm ³)
19	2.831	2.680	2.591
16	2.818	2.690	2.705
13.2	2.890	2.743	2.672
9.5	2.909	2.719	2.629
4.75	2.898	2.685	2.607
2.36	2.804	—	—
1.18	2.788	—	—
0.6	2.779	—	—
0.3	2.789	—	—
0.15	2.842	—	—
0.075	2.872	—	—
Filler		2.803	
requirements		>2.6	

Table 2. The main performance index of high viscosity asphalt

Tests item		unit	results	specifications	testing methods
Penetration(25℃,100g,5s)		0.1mm	43	≥40	T 0604—2011
Softening point (global method)		℃	104.0	≥80	T 0606—2011
Ductility ;	(15℃)	cm	60.4	≥50	T 0605—2011
	(5cm/min) (5℃)	cm	22	≥20	
Viscosity (60℃)		Pa.s	41000	≥20000	T 0620—2000
Tenacity (25℃)		N.m	39.47	≥20	T 0624—2011
After TFOT hangover	Mass loss	%	—	≤0.6	T 0609—1993
	penetration ratio	%	86.5	≥65	T 0604—2000

Gradation selection

The gradation of OGFC-13 mixture was determined according to the "technical specification for construction of highway asphalt pavement" (JTG F402004). However, due to the gradation of OGFC-20 mixture was not recommended in the specification, the gradation was referring to the relevant requirements of research experience (Lv 2010). The 2.36mm passing rate were selected as control sieve sizes. Three gradation types of aggregates with different 2.36mm passing rate were determined, respectively. The gradations of OGFC-13 and OGFC-20 were shown in the Figure 1 and Figure 2.

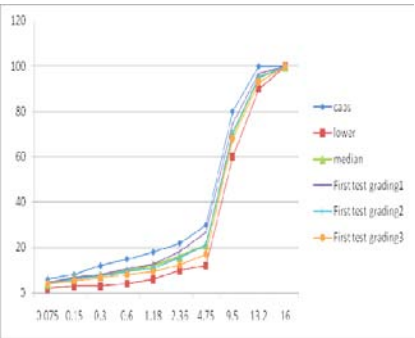


Figure 1. OGFC-13 initial grading curve

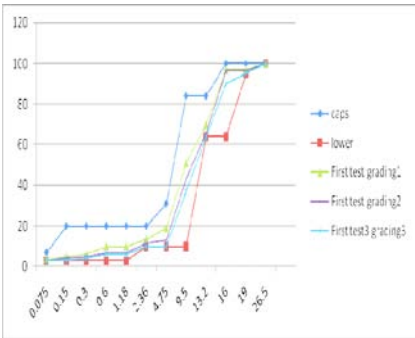


Figure 2. OGFC-20 initial grading curve

The Marshall test was conducted for the three kinds of first gradation. According to Marshall Test results, the relationship between air void and 2.36mm passing rate was shown in Table 3 and Figure 3. The air void and the passing rate of

2.36mm have a good linear relationship and the linear correlation equation was calculated.

Table 3. Air void and 2.36mm through the relation table rate

project		First gradeNO.1	First gradeNO.2	First gradeNO.3
OGFC-13	Passing rate of 2.36mm (%)	18.5	15.5	12.5
	The air void of the specimens (%)	16.4	20.2	22.4
OGFC-20	Passing rate of 2.36mm (%)	13.5	11.5	10
	The air void of the specimens (%)	19.8	23.7	25.5

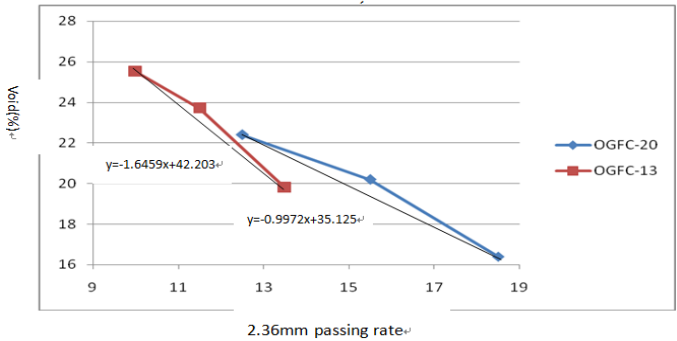


Figure 3. The relationship between 2.36mm passing rate and air void

The 2.36mm passing rates were calculated for 18%, 21%, 24% target air void with the linear correlation equation (Li 2002). The 2.36mm passing rates were 19.2%, 16.4%, 12.5% for OGFC-13 and 14.5%, 12.8%, 11.2% for OGFC-20. The adjusted gradation of aggregates was shown in Table 3 and Table 4.

Table 4. OGFC-13 gradation of aggregates

Gradation types	Passing of the sieve (mm) %									
	0.075	0.15	0.3	0.6	1.18	2.36	4.75	9.5	13.2	16
18% air void	5	6.7	8.4	10.9	13.5	19.2	27.9	74.4	97	100
21% air void	4.5	6.2	7.7	9.8	11.6	16.4	22.8	72	95.7	100
24% air void	4.2	5.4	6.8	8.1	9.4	12.5	17.2	68	93	100

Table 5. OGFC-12 gradation of aggregates

Gradation types	Passing of the sieve (mm) %											
	0.075	0.15	0.3	0.6	1.18	2.36	4.75	9.5	13.2	16	19	26.5
18% air void	3.5	6	8	12	12	14.5	21	51	73	97	97	100
21% air void	3	5	6	9	9	12.8	17	43	69	96	96	100
24% air void	3	4	5	7	7	11.2	14	37	67	90	95	100

Optimum asphalt content

The leakages of mixtures were measured to determine the optimum asphalt content. The optimum asphalt was determined for three kinds of mixtures with different air void, respectively. The relationship between the ratio of asphalt leakage loss and asphalt content, in intervals of 0.3%, were drawn. The optimum asphalt content of OGFC-13 were 4.8%, 4.5% and 4.2%. The optimum asphalt content were 4.8%, 4.4% and 3.9% for OGFC-20.the tests results were shown in Table 6 and table 7. All the indexes meet the specification requirements.

Table 6. The index mixture of OGFC-13

Test item	bulk density (g/cm3)	Air void (%)	Connected air void (%)	Asphalt leakage loss (%)	scattering loss (%)
Result	2.10	18.49	14.32	0.17	12.70
	2.02	21.69	17.44	0.26	14.71
	1.99	23.84	18.98	0.18	18.95
The standard value	—	18-25	—	<0.3	<20
Experimental method	T 0708	T 0708	T 0708	T 0732	T 0733

Table 7. The index mixture of OGFC-20

Test item	bulk density (g/cm3)	Air void (%)	Connected air void (%)	Asphalt leakage loss (%)	scattering loss (%)
Result	2.13	17.60	12.51	0.28	5.50
	2.06	21.61	16.12	0.12	9.40
	2.01	23.80	20.11	0.15	17.30
The standard value	——	18-25	——	<0.3	<20
Experimental method	T 0708	T 0708	T 0708	T 0732	T 0733

PERFORMANCE TEST

According to china specification "highway engineering asphalt and asphalt mixture test procedures" (JTJ E20-2011), the Marshall Test results were shown in Table 8. The Marshall stability decreased with the increase of air void smaller gradually. And the stability of OGFC-20 mixture was significantly higher than OGFC-13 with same air void.

Table 8. The results of Marshall test

test index	air void (%)	Marshall stability (KN)	The standard value (KN)	Experimental method;
The result of OGFC-13	18.49	5.56	≥3.5	T 0709
	21.69	4.23	≥3.5	T 0709
	23.84	3.54	≥3.5	T 0709
The result of OGFC-20	17.6	7.00	≥3.5	T 0709
	21.6	5.81	≥3.5	T 0709
	23.8	5.10	≥3.5	T 0709

The wheel tracking tests were conducted for the OGFC-13 and OGFC-20 mixtures according to "highway engineering asphalt and asphalt mixture test procedures" (JTJ E20-2011). The method was used to evaluate the high temperature resistance of the mixture in china. The dynamic stability was shown in table 9. The data shows that, with the increase of air void, dynamic stability value becomes smaller gradually. The dynamic stability of OGFC-20 mixture was significantly higher than OGFC-13. This means that the OGFC-20 mixture has better high temperature resistance than that of OGFC-13.

Table 9. Test results of high temperature stability

Test index	air void (%)	Dynamic stability (t/mm)	The standard value	Experimental method;
OGFC-13	18.49	11667	≥3000	T 0719
	21.69	10161	≥3000	T 0719
	23.84	8076	≥3000	T 0719
OGFC-20	17.6	12429	≥3000	T 0719
	21.6	11998	≥3000	T 0719
	23.8	9538	≥3000	T 0719

In addition, the permeable test was conducted for OGFC-13 and OGFC-20 mixtures. The results were shown in Table 10. The relationship between permeability coefficient and air void were shown in Figure 4 and Figure 5.

Table 10. The results of permeable test

Test index	air void	Connected air void	Permeability coefficient Cw (ml/s)
The result of OGFC-13	18.50%	14.31%	66.23
	21.70%	17.32%	83.33
	23.80%	18.98%	93.02
The result of OGFC-20	17.61%	12.50%	28.92
	21.62%	16.11%	54.64
	23.84%	20.13%	90.50
Empty state	100%	100%	112.36

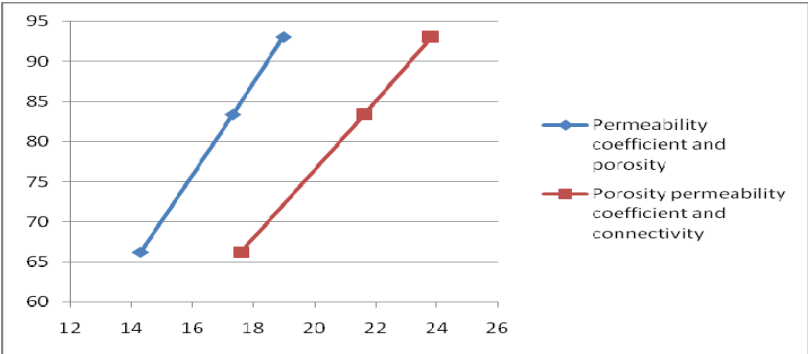


Figure 4. The relationship between permeability and air void of OGFC-13

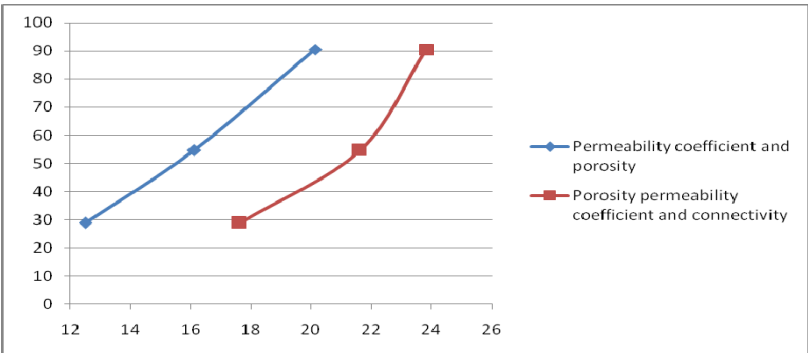


Figure 5. The relationship between permeability and air void of OGFC-13

Based on the above results, to balance the performance and permeability of OGFC asphalt mixture, the OGFC-20 with 21% air voids is recommended for wearing course of flexible pavement.

CONCLUSIONS

- (1) With the increase of air void, Marshall Stability gradually decreased. The stability of OGFC-20 mixture was significantly higher than OGFC-13 with same air void.
- (2) The dynamic stability of OGFC-20 was higher than that of OGFC-13. It means that the OGFC-20 has better high temperature resistance than OGFC-13, due to the coarse aggregate forms a better skeleton structure.
- (3) A good linear relationship were shown between the air void (greater connectivity air void) and permeability coefficient.

(4) To balance the performance and permeability of OGFC asphalt mixture, the OGFC-20 with 21% air voids is recommended for wearing course of flexible pavement.

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Laboratory Evaluation of Decontamination Ability for Runoff Water through OGFC Asphalt Mixtures with Different Air Voids

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ABSTRACT

OGFC (open graded friction course) asphalt mixture is a permeable mixture allowing water to flow through the pavement and drain out to the sides. The friction course significantly reduces splash/spray from vehicles and reduces vehicle noise. However, the accumulated pollutants in the surface of pavement, caused by vehicle fuel leak, tire wear, and vehicle exhaust adhesion can drain into natural water. It can cause environmental water pollution. In this study, the decontamination ability of OGFC with different air voids was investigated. The high temperature resistance of OGFC asphalt mixture was evaluated by wheel tracking testing. A new rain simulation device was designed to control the rainfall intensity and duration in laboratory. The density of COD (chemical oxygen demand) and SS (suspend solid) were measured. The results showed that the decontamination ability increased with the increasing of air voids for OGFC asphalt mixture and the decontamination effect for COD was less than that for SS. Furthermore, to balance the performance and decontamination effect, the OGFC asphalt mixture with 21% air voids was recommended for wearing course of pavement.

INTRODUCTION

In recent years, with the rapid urbanization process, the natural ground is gradually substituted by the impermeable artificial structures. Furthermore, urban road and affiliated facilities account for more than 25% of the impermeable areas (Wang et al. 2007). Related studies have shown that the surface runoff is the main water pollution source of the city, only less than the city raw sewage and industrial waste water. The pollutant of the leaking fuel of driving motor vehicles and tire abrasion can accumulate on the surface of the road. Then pollutants flow into the natural water under the washed out effect of rain, which causes the pollution of the natural water (Chen 2004).

To overcome the problems mentioned above, the Low Impact Development (LID) has been mentioned in recent years to reduce the influence of urban development on the ecological environment (Scott and Carter 2011). One of the main methods of the application of LID in storm water management and control is the

adoption of permeable pavement in order to achieve the source control of runoff pollution. OGFC (open graded friction course) asphalt mixture is one of bituminous materials of wearing course for flexible pavement. The permeable mixture allows water to flow through the pavement and drain out to the sides. In addition, OGFC pavement is very effective to the removal of storm water runoff pollutants. Utilizing its feature of macrospore, the rain on road surface can timely penetrate in the road and be purified.

In this research, the decontamination ability of OGFC with different air voids (18%, 21%, and 24%) was investigated. The OGFC specimens were designed using Marshall mix design procedure. A new rain simulation device was designed to control the rainfall intensity and duration in laboratory. The density of COD (chemical oxygen demand) and SS (suspend solid) were measured. The optimal air void content was chosen from the perspective of balancing between the road performance of mixture and the runoff pollution control effect.

MATERIALS DESIGN

Raw materials. In this research, coarse aggregates were basalt. The fine aggregate was limestone. The density results are shown in Table 1. The main performance index properties of high viscosity asphalt are summarized in Table 2.

Table 1. Mineral aggregate density

Aggregate size (mm)	Apparent relative density (g/cm ³)	The relative density of bulk volume (g/cm ³)	The relative density of dry (g/cm ³)
19	2.831	2.680	2.591
16	2.818	2.690	2.705
13.2	2.890	2.743	2.672
9.5	2.909	2.719	2.629
4.75	2.898	2.685	2.607
2.36	2.804	—	—
1.18	2.788	—	—
0.6	2.779	—	—
0.3	2.789	—	—
0.15	2.842	—	—
0.075	2.872	—	—
Filler		2.803	
requirements		>2.6	

Table 2. The main performance index of high viscosity asphalt

Tests item	unit	results	specifications	Testing methods
Penetration(25℃,100g,5s)	0.1mm	43	≥40	T 0604—2011
Softening point (global method)	℃	104.0	≥80	T 0606—2011

Ductility ;	(15℃)	cm	60.4	≥50	T 0605—2011
(5cm/min)	(5℃)	cm	22	≥20	
Viscosity (60℃)		Pa.s	41000	≥20000	T 0620—2000
Tenacity (25℃)		N.m	39.47	≥20	T 0624—2011
After TFOT hangover	Mass loss	%	—	≤0.6	T 0609—1993
	penetration ratio	%	86.5	≥65	T 0604—2000

Gradation selection. The 2.36mm passing rate was used as the control sieve. The gradation of OGFC-13 are shown in Table 1 and Fig. 1.

Table 3. The first grade of OGFC-13

Gradation types	Through the sieve (mm) of quality /%									
	0.075	0.15	0.3	0.6	1.18	2.36	4.75	9.5	13.2	16
upper limit	6	8	12	15	18	22	30	80	100	100
lower limit	2	3	3	4	6	10	12	60	90	100
median	4	6.5	7.5	9.5	12	16	21	70	95	100
First gradeNO.1	4.9	6.6	8.2	10.6	13	18.5	26.8	75	97	100
First gradeNO.2	4.4	6	7.4	9.4	10.8	15.5	21.2	71	95	100
First gradeNO.3	4.2	5.4	6.8	8.1	9.4	12.5	17.2	68	93	100

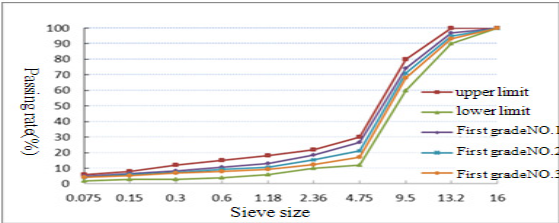


Figure 1. The first grading curve of OGFC-13

For three initial test gradations, the Marshall specimens were prepared and the air voids of the specimen were measured. The relationship between air voids and 2.36mm passing rate was shown in Fig. 2. There is a good linear relationship between air voids and 2.36mm passing rate. A linear equation was obtained between them.

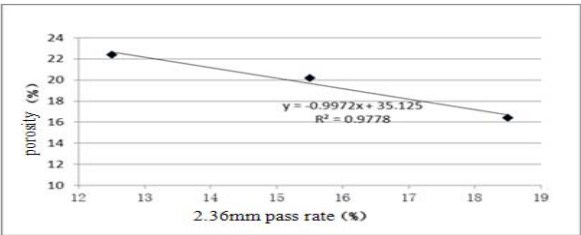


Figure 2. Relationship between air voids and 2.36mm passing rate

According to linear correlation equation, we came to the conclusion that the passing rate of air voids of 18%, 21% and 24% is respectively 19.2%, 16.4% and 12.5%. Adjusted design gradation and curve are shown in Table 4 and Fig. 3.

Table 3. The adjusted design gradation of OGFC-13

Gradation types	Passing of the sieve (mm) %									
	0.075	0.15	0.3	0.6	1.18	2.36	4.75	9.5	13.2	16
18% air void	5	6.7	8.4	10.9	13.5	19.2	27.9	74.4	97	100
21% air void	4.5	6.2	7.7	9.8	11.6	16.4	22.8	72	95.7	100
24% air void	4.2	5.4	6.8	8.1	9.4	12.5	17.2	68	93	100

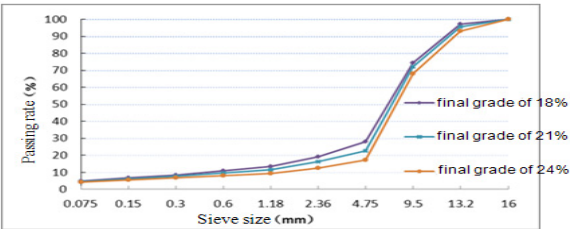


Figure 3. The final grading curve of OGFC-13

Optimum asphalt content

The optimum asphalt content is determined by leakage test analysis: every 0.3% an interval, drawing “analysis of leakage losses-asphalt content” curve and then tangents of the curve around the inflection point, finding the point of intersection of tangents as inflection point –that is the best asphalt content, see Fig. 5, Fig. 6 and Fig. 7, respectively.

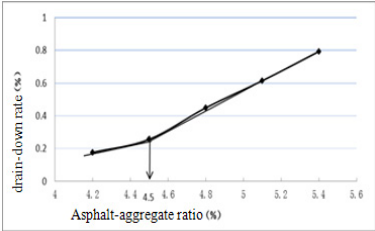
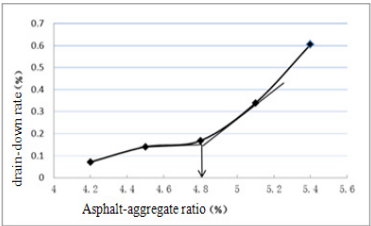


Figure 4. relationship between asphalt content and the loss of drain-down for mixture with 18% air void

Figure 5. relationship between asphalt content and the loss of drain-down for mixture with 21% air void

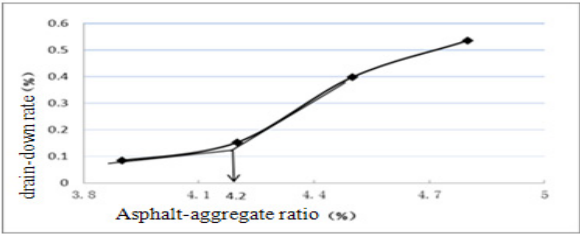


Figure 6. relationship between asphalt content and loss of drain-down for mixture with 24% void

Based on the determined final gradation and the optimum asphalt content, the Marshall specimens were prepared. The results were shown in Table 5.

Table 5. The index mixture of OGFC-13

Test item ;	bulk density (g/cm3)	Air void (%)	Connected air void (%)	asphalt leakage loss (%)	Fort Kent scattering loss (%)
Result	2.10	18.49	14.32	0.17	12.70
	2.02	21.69	17.44	0.26	14.71
	1.99	23.84	18.98	0.18	18.95
The standard value	—	18-25	—	<0.3	<20
Experimental method	T 0708	T 0708	T 0708	T 0732	T 0733

Performance test

A series of laboratory tests were conducted for OGFC-13 with three different air void, Including Marshall test, water stability test, rutting test, etc. Test results were shown in Table 6.

Table 6. Performance test results of OGFC - 13 mixtures

test index	Air void (%)	Marshall stability (KN)	residual stability (%)	Dynamic stability(t/mm)
Result	18.49	5.56	87.2	11667
	21.69	4.23	85.5	10161
	23.84	3.54	81.4	8076
The standard value	18-25	≥3.5	≥80	≥3000
Experimental method	T 0708	T 0709	T 0709	T 0719

In addition, testing the water penetration for three different air voids of asphalt mixture, the results of water permeability coefficient are shown in Table 7. The relationship between Water permeability coefficient and connected air void or air void were shown in Fig. 7 and Fig. 8. Through the chart can be seen that with the increment of air void and effective air void, coefficient of permeability also increasing, and air void, effective air void and permeability coefficients showed a good linear correlation. The state of built on stilts measured permeability coefficient has played a good reference function.

Table 7. The result of water penetration test

air void	connected air void	Water permeability coefficient Cw(ml/s)
18.50%	14.3%	66.23
21.70%	17.3%	83.33
23.80%	18.98%	93.02
state of built on stilts	100%	112.36

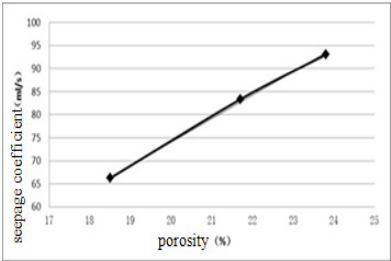


Figure 7. The relationship curve of water permeability coefficient and air void

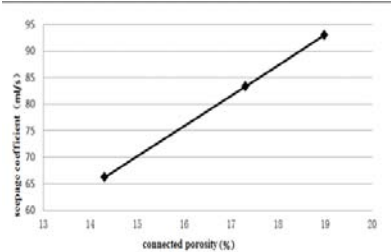


Figure 8. The relationship curve of water permeability coefficient and connected air

EVALUATE THE EFFICACY OF THE RUNOFF POLLUTION CONTROL

The selection of pollutants and the preparation of the samples

Road runoff pollutants ingredients include solid particles suspended matter SS, organic pollutant COD (chemical oxygen demand (COD) for characterization of index), heavy metals, nitrogen (N) and phosphorus (P) of pollutants, chloride salts, etc. Among them, Solid particles suspended matter SS, COD organic pollutant is the main pollutants in storm water runoff. Studies have shown that (Zheng and Jia 2007), there exists a good linear correlation between Road runoff pollutants that including COD, total lead (Pb), total zinc (zinc) and SS. Therefore, this study selected runoff pollutants are solid particles suspended matter SS and COD organic pollutant.

Suspended solids in water samples be prepared by the surface sedimentary soil, organic pollutant COD is composed of humic acid. We selected a number of

sampling points of 1m² on the collector road of Beijing downtown, and then cleaning the sedimentary soil on the road. According to the study of particle size distribution in Beijing area (Chen 2011), SS size are located under 75 microns. Therefore, the sedimentary soil was sieved with 75 microns standard screen and the soil sample passing through 75 microns diameter was dried and saved. According to the survey results of initial runoff pollutants concentration in Beijing, prepared solid suspended solids that its goals concentration is 1200 mg/L. The goal concentration of organic pollutant COD is 800 mg/L. According to the simulated rainfall test, the author calculated the quality of pollutants and prepared them.

Experimental method

In order to study that OGFC material of different air void influence on the removal efficiency of SS and COD, We developed the test device to simulate the natural rainfall, and processed the corresponding specimen mold. The main components of test equipment include round tank, small mixer, flow meter, valve, showerhead Etc.



Figure 9. Water purification device

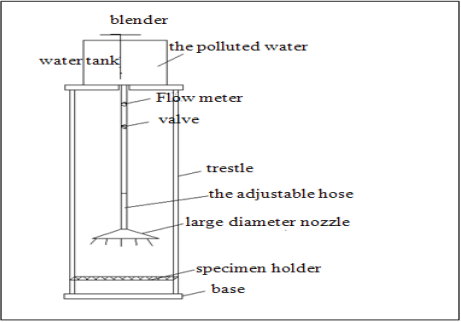


Figure 10. Design of water purification device

During the experiment, the polluted water of target concentration (rain) first mixed evenly by the blender, then through the valve control to adjust the rainfall intensity, the polluted water filtering through the OGFC mixture specimen ,finally, collected the filtered water. The overall water purification device object diagram as shown in figure 9, Water purification device was shown in figure 10.

Through controlling two factors which the rainfall intensity and duration to simulate natural rainfall (Xie et al. 209). Using a constant rainfall intensity that is 110 mm/h, simulating the heavy rain intensity to test pollutants purification effect of OGFC pavement. This study simulated early runoff time for 20 min, sampling once every four minutes, the rain is the polluted water that prepared. Later period rainfall time is 50 min, sampling once every ten minutes, the rain is pure water. Collecting the original water samples which has not been filtered and filter water of sampling time point, respectively determined the SS concentration and COD concentration of water

samples.

Air void influences on the purification evaluation of polluted runoff

Respectively forming the specimen according to the different air void which 18%, 21%, and 24%. After remolding put in the mold on standby. Open the blender, regulating valve, In accordance with the test method measured the SS and COD concentration of specified time water samples. According to the following formula to calculate pollutant adsorption rate of K, and obtain the relationship between air void and adsorption rate, as shown in table 8.

$$K = (Q_{\text{original}} \times 20 - \sum Q_t \times 4 - \sum Q_{t'} \times 10) / (Q_{\text{original}} \times 20) \tag{1}$$

Q_{original} ——SS (COD) concentration of the original water samples

Q_t ——t time SS concentration (COD) in the early 20min;

$Q_{t'}$ ——t time SS concentration (COD) in the later period

Table 8. The relationship between air void and adsorption rate

air void	SS	COD
	adsorption rate	adsorption rate
18.50%	58.94	57.12
21.70%	61.83	63.41
23.80%	73.67	67.55

As is shown in Table 8, under constant rain rainfall intensity, permeable road surfaces with different porosity have significant removal effect to suspended solids SS. With the increase of the air void, the adsorption rate for SS is getting higher, and the purifying effect of air void of 24% is obviously better than the other two porosities. The permeable pavement adsorption rate of COD of different air void and SS suggested a similar rule, but the data shows that the effect of purification of COD is poor than the SS.

Based on the above test results, the higher air void, the worse Marshall Stability, water damage and other road performance of OGFC-13-type mixture, the greater the scattering loss, the better water purification. When the air void is 24%, scattering loss is 19%, approaching 20% of specification limit, and Marshall Stability is 3.54, meeting regulatory requirements exactly. When the air void is 18%, permeability coefficient is relatively low, and water permeability is relatively poor. When under heavy rain intensity, its water purification function is poor. So, from the respect of balance design of air void, mechanical properties of mixture, permeable and water purification, we choose 21% to be the best air void of OGFC-13.

CONCLUSION

(1) The performance properties of OGFC-13-type mixture, the high temperature stability, and moisture resistance, decrease with increasing the air void. However, under heavy rainfall intensity, it is better for the purification of road runoff pollutants. Considering the balance between mixture performance properties and control effect of road runoff pollution, the 21% air void was recommended.

(2) With the increasing of air void and effective air void, permeability coefficient increases. Meanwhile, air void, effective air void, and permeability coefficients showed good linear correlations.

(3) Under constant rain rainfall intensity, OGFC-13 with different air void are effective to the removal of SS and COD. The purifying effect of air void of 24% is better than the other two. Rainfall intensity plays an important role in the water purification effect of large pore asphalt mixtures. The relationship between rainfall intensity and water purification effect needs to be studied in the future.

ACKNOWLEDGEMENT

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Performance Evaluation of High RAP Asphalt Mixture with Fiber and Warm Mixing Agent

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ABSTRACT

The WRAM (warm recycled asphalt mixture) is a new type bituminous material for the green environmental friendly pavement construction. The WRAM technology combines the benefits of hot recycled and warm mix. It significantly reduces energy consumption and gas effusion due to lower mixing and compact temperature. However, related research indicated that low temperature and water stability of WRAM with high RAP content (more than 30% by weight) cannot meet the requirements of the specification. In this research, the fiber was applied to improve the performance of AC-20 WRAM with 40% and 50% RAP content. The composition design was carried out using Marshall Test. The optimum bitumen content is 4.9% and 5.1%, respectively. A series of laboratory tests, such as wheel tracking before and after conditioning, and beam bending at low temperature, were carried out to evaluate the performance properties of AC-20 WRAM with reinforcing fiber. It was shown that this method can effectively improve the low temperature and water stability of asphalt concrete pavement. The properties of WRAM meet the requirements of the specification.

INTRODUCTION

At present, due to pavement repair and maintenance, a large number of road reconstruction produce huge amount of waste asphalt in China. Reclaimed asphalt pavement (RAP) contains aged bitumen (approximately 5% by weight) and variety of particle size aggregates (about 95%). If the RAP cannot be recycled, it not only cause huge waste of resources, but also pollute to the environment. It does not meet the basic national policy of sustainable development (Qin 2006). The hot mix plant recycling technology is popularly used in China. However, the content of RAP is 10 -30% in recycled asphalt mixture, while consuming a lot of fuel and emit large quantities of gas (Qin et al. 2006). Warm mix asphalt technology is a new type of asphalt technology. It mainly reduces the viscosity of the asphalt and improves asphalt high temperature fluidity by adding warm mix material. It makes asphalt and mineral aggregate to be mixed at a lower temperature, named as warm mixing asphalt (WMA). Because of its lower mixing and compaction temperatures, its fuel

consumption decreases by 20% to 50%. It also effectively reduces the aging of asphalt and enhances the life of the road compared to hot mix asphalt (Malliek et al. 2007). In addition, it reduces emissions and pollution to improve the working environment, especially the working environment of tunnels and underground transportation and other enclosed spaces. Warm mix asphalt can substantially all exist equipment of hot mix asphalt, without transformation of production machinery and equipment.

WMA technology combines warm mix with renewable technologies to make up the traditional hot asphalt recycling technology insufficiency, achieve the recycling of waste asphalt and energy saving double effect, and extend the pavement life. It is a revolutionary asphalt pavement materials technology, which epitomizes the advantages of the use of waste and energy conservation. It was found that if AC-20-type warm mix recycled asphalt mixture contains more than 30% the amount of aged material at low temperature, water stability and other road performance do not meet the regulatory requirements. Lignin fibers can have effect on asphalt toughening, thickening effect, enhance bond strength of aggregate particles, mortar shear, tensile strength, and improve low temperature properties and water stability of mixed families. Therefore, this study improved road performance of AC-20-type warm mix recycled mixtures containing 40% and 50% aged material by adding fiber in order to improve the ratio of aged material blending.

RAW PERFORMANCE TEST

Mineral aggregate

This test used mineral aggregate 0 ~ 4.75mm diameter of limestone, 4.75 ~ 19mm diameter of basalt. Aggregates requires clean, dry, rough surface, In accordance with the test method in "Highway Engineering Aggregate testing procedures" (JTG E42-2005), the densities of mineral aggregate were tested (JTJ058 2010). To reduce errors, the asphalt mixture gradation was closer to synthesize gradation. The mineral aggregate were stored according to each sieve for screening.

Asphalt

Penetration grade 70 # virgin asphalt was applied in this research. According to "technical specification for construction of highway asphalt pavement (JTG F40-2004)" and "regulations of highway engineering asphalt and asphalt mixture test (JTJ 052-2000) to evaluate the properties asphalt and its conventional index test results are listed in Table 1-1.

Table 1-1. 70 # bitumen properties test results

Item	Unit	Requirements	Results	Test method
Penetration (25℃,5s,100g)	0.1mm	60~80	74	T 0604—2000
Softening point TR&B	℃	≥46	50.2	T 0606—2000

Ductility (15℃,5cm/min)	cm	≥100	>100	T 0605—1993

Fiber

In this study the lignin fibers, lignin fiber reinforced the role played in the mixture to improve high-temperature mixture rutting resistance and fatigue resistance capability, so that the road of high temperature stability and fatigue durability is improved. Lignin fibers continued to show flexibility at low temperatures, which can effectively resist thermal stress, reduce temperature shrinkage cracks and improve the pavement temperature crack resistance. Different types of asphalt mixture, lignin fiber content in 0.25% ~ 0.4% can effectively improve road performance.

According to experience, the study selected lignin fiber content is 0.3%. This study used loose lignin fibers, performance testing indicators in Table 1-2.

Table 1-2. Lignin fibers test results

Indicators	Unit	Test results	Technical requirements
Ash content	%	20.1	18±5, No volatiles
PH value	--	7.5	7.5±1.0
Absorption rate	%	5.0	Mass of fiber 5.0 ± 1.0-fold
Moisture	%	3.0	<5%(By mass)
Density	%	1.2	--

Warm mixing agent

In order to recycling the mixture and reduce the mixing and compaction temperatures, a new warm mixing agent, named as LTR, was applied in this research.

LTR is a liquid at room temperature, a viscous insoluble in water, soluble in aromatic hydrocarbons and alcohol. LTR is a surface active agent which is added to reduce the mixing of asphalt and aggregates the resistance, thereby reducing the mixture mixing and compaction temperatures. Add a small amount of the LTR, using the weight ratio of the asphalt is only 0.2% to 0.5% added, it can be directly added to the bitumen or asphalt and stone mixing step (Larsen and Robertus 2005). LTR on the asphalt itself, the physical and chemical properties less affected, but the level of performance of the asphalt itself will not cause change, you can keep a good traditional hot mix asphalt physical and mechanical properties (JTJ052 2000). Based on product performance, determine the viscosity drag reduction agent used was 0.5%, and its physical and chemical indicators in Table 1-3.

Table 1-3. LTR physical performances

Items	Unit	results
Physical state (20℃)	—	Liquid state
Freezing point	℃	-10
Flash point	℃	178
Density (25℃)	g/cm ³	1
Solubility	—	Soluble in aromatic, alcohol insoluble in water

MIXTURE DESIGN

This paper used the Marshall Mix design method of recycling temperature and mixture ratio design, Marshall Compaction method for molding specimens for research related to performance. According to the selected grading type, we determine the scope of mineral aggregate gradation. Selecting the appropriate grading scale, according to the Marshall specimen volume parameters, determine warm mix asphalt recycling mixture of the best proportion, warm mix asphalt recycled asphalt amount includes new asphalt and old aged asphalt compound the original two-part .

Aggregate gradation

According to "asphalt pavement construction specifications" (JTG F40-2004) coupled with lower-level requirements, this article uses the coarse and fine aggregate sieve into a file by size, for aggregate gradation design. AC-20 asphalt mixture composed of aggregate gradation shown in Figure 1 AC-20 asphalt mixture aggregate gradation

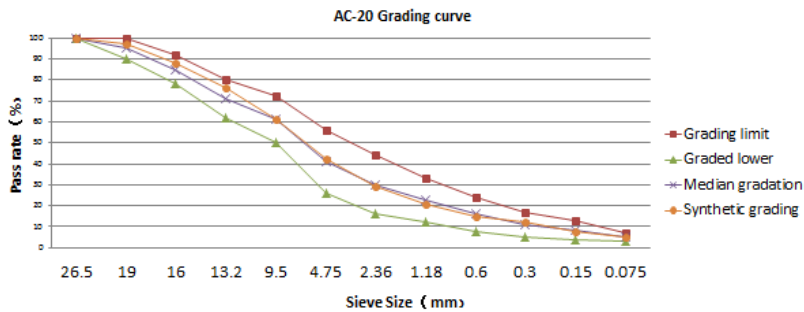


Figure 1. AC-20 synthetic grading curve

AC-20 Warm Mix Asphalt with 40% RAP

Optimum asphalt content

Preparing Marshall Specimens, based on past experience, an initial 3.5%, 4%, 4.5%, 5%, 5.5% five kinds of Binder content, lignin fiber content is 0.3%. Accordance with the ordinary hot-mix asphalt request preparing specimen volume

determination Marshall specimens for Marshall test parameters and determine its stability and flow values, the results shown in Table 2-1.

Table 2-1. AC-20 test results

Binder content (%)	Bulk density (g/cm ³)	stability (kN)	Flow value (mm)	Air void (%)	VMA (%)	VFA (%)
3.5	2.39	5.92	2.11	9.26	16.40	43.56
4.0	2.41	7.06	2.55	7.92	15.97	50.42
4.5	2.43	8.07	2.64	6.28	15.54	59.60
5.0	2.45	8.27	3.12	4.50	15.44	70.86
5.5	2.45	7.47	3.27	2.75	15.87	78.05
specification	—	≥8	1.5~4	4~6	≥13	65~75

AC - 20 calculated from above, the optimum proportion of asphalt mixture is 4.9%, the ratio of the Marshall test results are shown in Table 2-2.

Table 2-2. The optimum proportion corresponding index test results

Binder content (%)	bulk density (g/cm ³)	stability (kN)	flow value (mm)	Air Void (%)	VMA (%)	VFA (%)
4.9	2.45	8.83	3.0	4.88	15.53	68.57
specification	—	≥8	1.5~4	4~6	≥13	65~75

Temperature mixing materials performance test

(1) High temperature stability performance test

According to the above test to determine the optimum proportion of high temperature stability performance inspection, test results such as Table 2-3.

Table 2-3. High temperature stability test results

Binder content (%)	Dynamic stability (pass/mm)	specification
4.9	3319	≥1000

(2) Water stability performance test

According to the above test to determine the optimum proportion of preparing Marshall Specimen, water stability test, freeze-thaw splitting test and immersion Marshall test, test results such as Table 2-4.

Table 2-4. Water stability test results

Binder content (%)	4.9	
test item	The soaking stability (%)	Freeze-thaw splitting strength ratio (%)
	87.2	77.5
Specification values	≥80	≥75

(3) Low temperature performance test

According to the above test to determine the optimum proportion of the molding plate specimen with wheel grinding method, and cut into trabecular, the UTM - 25 material testing machine for testing, test temperature for - 10 °C, the results in Table 2-5.

Table 2-5. Low temperature bending test results

Mixture type	Binder type	Failure strain (με)	
		Measured	Specification
AC-20	70#	2326.5	≥2000

AC-20 Warm Mix Asphalt with 50% RAP

Optimum asphalt content

Molding Marshall Specimen, based on past experience, the initial 3.5%, 4%, 4.5%, 5%, 4.5%, five kinds of binder content, lignin fiber content is 0.3%. Molding according to the requirements of ordinary hot mix asphalt mixture specimen, determine the volume parameter Marshall specimen and Marshall test to determine the degree of stability and flow values, the results in Table 2-6.

Table 2-6. Adding 50% RAP AC - 20 test results

Binder content (%)	Bulk density (g/cm ³)	stability (KN)	flow value (mm)	Void fraction (%)	VMA (%)	VFA (%)
3.5	2.38	7.28	2.29	9.86	16.49	40.20
4.0	2.42	8.11	2.67	8.08	15.72	48.59
4.5	2.42	9.02	2.81	6.76	15.89	57.45
5.0	2.45	12.46	3.16	4.75	15.51	69.37
5.5	2.44	9.77	3.24	4.50	16.09	72. 04
specification	——	≥8	1.5~4	4~6	≥13	65~75

AC - 20 calculated from above, the optimum proportion of asphalt mixture is 5.2%, the ratio of the Marshall test results are shown in Table 2-7.

Table 2-7. The optimum proportion corresponding index test results

Binder content (%)	Bulk density (g/cm ³)	stability (kN)	flow value (mm)	Void fraction (%)	VMA (%)	VFA (%)
5.2	2.47	8.9	3.6	4.13	14.91	72.82
specification	—	≥8	1.5~4	4~6	≥13	65~75

Temperature mixing materials performance test

(1) High temperature stability performance test

According to the above test to determine the optimum proportion of high temperature stability performance inspection, test results such as Table 2-8.

Table 2-8. High-temperature stability test results

Binder content (%)	Dynamic stability (times/mm)	Specification
5.2	4000	≥1000

(2) The water stability performance test

According to the above test to determine the optimum proportion of preparing Marshall specimen, water stability test, freeze-thaw splitting test and immersion Marshall test, test results such as Table 2-9.

Table 2-9. Water stability test results

Binder content (%)	5.2	
test item	The soaking stability (%)	Freeze-thaw splitting strength ratio (%)
	86.8	75.7
Specification values	≥80	≥75

(3) The low temperature performance test

According to the above test to determine the optimum proportion of the molding plate specimen with wheel grinding method, and cut into trabecular, the UTM - 25 material testing machine for testing, test temperature for - 10 °C, the results in Table 2-10.

Table 2-10. Low temperature bending test results

The mixture type	Asphalt type	Failure strain (με)	
		The measured values	Specification values
AC-20	70#	2219	≥2000

CONCLUSIONS

This paper studied the performance properties of fiber reinforcing WRAM with 40% and 50% RAP contents, such as high temperature stability, water stability performance, and low temperature crack resistance, etc.. Following conclusions were obtained:

- (1) With the increase of the proportion of aged material, mixing temperature of recycled asphalt mixture, Marshall Stability and dynamic stability are increased. The fiber added can effectively improve the rutting resistance of the mixture.
- (2) The freeze-thaw splitting strength ratio and Marshall stability of WRAM can satisfy the requirements of the specification. Warm mix of recycled asphalt mixture water stability was improved with fiber.
- (3) Lignin fiber increased the optimum asphalt content of mixture and the ductility of the mixture. The WRAM with lignin fiber has same cracking resistance as virgin materials at low temperatures.
- (4) Mixed with 40% of RAP, the optimum binder content was 4.9% for WRAM. It was 5.2% for WRAM with 50% RAP. It means that the optimum binder content increases with the increasing of the proportion of RAP.

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Study on the Performance of Direct Coal Liquefaction Residue Modified Mixture Asphalt

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ABSTRACT

The properties of the two different blends composed of DCLR (direct coal liquefaction residue) and a binder using two processing methods are studied in this paper. One processing method is to blend powdered DCLR and a binder, and the other is to blend molten DCLR and a binder. The optimum adding content of DCLR is determined using a PG system, and furfural extract oil is used to improve the low-temperature properties of the blends. The performance of DCLR-modified mixture asphalt with the two different blends is also studied in this paper. At first, the DCLR-13 was designed using the Marshall method. Second, the rutting test, freeze-thaw splitting test, and low-temperature bending test were conducted on the DCLR-13. Finally, the performance of the DCLR-13 with the two different blends were evaluated and compared. Results indicate that the performance of DCLR-modified mixture asphalt satisfies the specifications, and the processing method has an influence on the performance of DCLR-modified mixture asphalt.

INTRODUCTION

China is a country with rich coal and gas but little petroleum. At present, the explored coal reserves are 1.3983 trillion tons, ranking second in the world. However, petroleum resources are relatively rare in China. In 2012, the gap of supply and demand for petroleum in China was nearly 300 million tons, which results in China's dependence on imported petroleum, up to 60%. In order to relieve the shortage of petroleum resources in China, it has been an important energy strategy trend to produce petroleum from coal. The country's demand for liquid fuel can be basically met by means of coal liquefaction. Coal liquefaction can be classified by direct liquefaction and indirect liquefaction according to the processing method, among which direct coal liquefaction for petroleum brings in a higher conversion rate and lower cost. Therefore, it has witnessed a quick development.

In the direct coal liquefaction process, apart from liquid fuels such as

gasoline and diesel, the main byproduct liquefied residues are also produced, which accounts for 30% of total feed coal. In fact, the direct coal liquefaction residue (DCLR) includes 30–50% of heavy oil and binderene, which is similar with Trinidad lake binder (TLA, a natural modifier to improve the binder properties) in composition and some physical properties. Among non-polymer additives, with a good modification property, TLA has been widely applied all over the world. Many important projects in China also use TLA, e.g., Jiang-Yin Bridge, the western runway of the Beijing Capital International Airport, and the Zi-Yang section of the Chengdu-Chongqing Expressway. With the construction of expressways in China and improvement of common highways, the demand for TLA also has increased. However, the application of TLA is restricted since the TLA source is limited and import price is high.

DCLR is a substance most similar with TLA in properties. Hence, the study on new technology that the blends DCLR and a binder are secondarily developed to a binder modifier, which is used for replacing the TLA to improve the binder properties and has very important scientific significance and application value. On the one hand, DCLR, as a binder modifier, can significantly improve the binder properties, reduce the price of the modified binder (the DCLR price is equivalent to one-quarter of TLA), and promote the development of the binder industry of China; on the other hand, the use of binder modified by DCLR is of positive significance in environmental protection, reasonable use of limited resources, and increase of DCLR utilization.

PROPERTIES OF DCLR

DCLR used in this paper is from the direct coal liquefaction residue manufactured by Mongolia Branch of China Shenhua Coal to Liquid and Chemical Co., Ltd., and its properties are as shown in Table 1.

Table 1. Properties of DCLR.

Item	Density(g/cm ³)	Softening point(°C)	25 °C penetration(0.1mm)
Test results	1.23	170	2

PROCESSING METHOD FOR BLENDING DCLR AND BINDER

DCLR used in this paper is powder and blends with a binder using two different processing methods. Processing method I is as follows: first, heat the binder, and then add different amounts of DCLR into the melting binder directly. In order to ensure that the DCLR is fully blended with the binder, the blends composed of DCLR and binder are sheared with a shear apparatus for 1 hour at low speed under the temperature of 160 °C. Processing method II is as follows: heat the binder and DCLR to melting status, and then blend the molten DCLR and the binder in a certain proportion. To ensure that the DCLR is fully blended with the

binder, the blends composed of DCLR and binder are sheared with a shear apparatus for 1 hour at low speed under the temperature of 160 °C.

DETERMINE THE DCLR CONTENT

A pen 60-80 binder was used in this study. The DCLR content was 5%, 10%, 15%, and 20%. Second, the different content DCLR and pen 60-80 binder were blended using processing methods I and II. Finally, the properties of the blends were tested, including the 25 °C penetration, the softening point, and the 10 °C ductility.

From the test results, it was found that:

- 1) With both processing methods I and II, the 25 °C Penetration of the blends was decreased as the DCLR content increased. When the DCLR content was more than 15%, the 25 °C penetration of the blends was lower than 40 (0.1mm).
- 2) With both processing methods I and II, the softening point of the blends was increased as the DCLR content increased. When the DCLR content was higher than 15%, the softening points of the blends increased very slowly.
- 3) With both processing methods I and II, the 10 °C ductility of the blends decreased as the DCLR content increased. When the DCLR content was higher than 15%, the 10 °C ductilities of the blends were almost zero.

After the DCLR was blended with the binder, the low-temperature properties of the blends were sharply degraded; some furfural extract oils (FEO) were added into the blends to improve the low-temperature properties of the blends. When the 5% furfural extract oil was added into blends, the 10 °C ductilities of the blends were improved and met the specifications. This suggests that the properties of the blends were the best when the 15% DCLR and 5% FEO were blended with the binder. The test results are shown in Table 2.

Table 2. Properties of Blends (Adding 15%DCL and 5%FEO).

Items	25 °C penetration (0.1mm)	Softening point	10 °C ductility	135 °C viscosity	Residue after RTFOT		
					Loss of mass	Residual penetration	10 °C ductility Residual
Unit	0.1mm	°C	cm	Pa.s	%	%	cm
Method	38	57.3	3.3	0.49	0.07	61	1.2
Method	28	59	6.2	0.48	0.03	58	1.1

PERFORMANCE OF MIXTURE ASPHALT

The AC-13 DCLR modified mixture asphalt was designed using the blends as a binder. The optimum binder content was 5.4%. The performance of the mixture asphalt was evaluated by using the rutting test, the freeze-thaw splitting test, and the low-temperature bending test. To compare the performance of the DCLR-modified mixture asphalt, the control mixture asphalt based on the same gradation was also made. The gradation of AC-13 can be seen in Table 3.

Table 3. Gradation of AC-13.

Seive(mm)	Passing ratio(%)	Limit value(%)
16	100	100
13.2	96.5	90–100
9.5	74	68–85
4.75	57.2	38–68
2.36	45.5	24–50
1.18	29.6	15–38
0.6	20.9	10–28
0.3	13.4	7–20
0.15	9.4	5–15
0.075	6.8	4–8

High-temperature performance. The anti-rutting capacities of the mixture asphalts were conducted by using the rutting test according *Standard Test Methods of Bitumen and Bitumens Mixture for Highway Engineering* (JTG E20-2011) in China. Table 4 shows the test results of the mixture asphalt.

Table 4. Dynamic Stability of Mixture Asphalts.

Processing method	Dynamic stability (pass/mm)		Limit values
	<i>DCLR-modified mixture</i>	<i>Control mixture</i>	
Processing method I	4182.8	3183.7	>1000
Processing method II	4022.0		

According to *Technical Specifications for Construction of Asphalt Pavements* (JTG F40—2004), the rutting resistance of the DCLR-modified mixture asphalt met the specifications under the two processing methods. Compared to the control mixture asphalt, the DCLR-modified mixture asphalt’s dynamic stabilities increased 40%. This meant that the rutting resistance of the mixture asphalt could be improved by adding DCLR.

Low-temperature performance. The low-temperature bending test was conducted on the mixture asphalts. Table 5 shows the test results of the mixture asphalt.

Table 5. Failure Strains of Mixture Asphalts.

Processing method	Failure strain (μϵ)		Limit values
	<i>DCLR-modified mixture</i>	<i>Control mixture</i>	
Processing method	6500.9	1120	>2000
Processing method	5802.6		

According to *Technical Specifications for Construction of Highway Asphalt Pavements* (JTG F40—2004), the anti-cracking capacities of the

DCLR-modified mixture asphalt satisfied the specification requirements under the two processing methods. Compare to the control mixture asphalt, the DCLR-modified mixture asphalt’s failure strain increased 400%. This meant that the anti-cracking capacities of mixture asphalt had been increased after adding DCLR.

Water sensibility. The freeze-thaw splitting test was conducted on the mixture asphalt. Table 6 shows the test results of the mixture asphalts.

Table 6. TSR of Mixture Asphalts.

Processing method	TSR(%)		Limit values
	<i>DCLR-modified mixture</i>	<i>Control mixture</i>	
Processing method I	85.4	90.1	>75
Processing method II	91.6		

According to *Technical Specifications for Construction of Highway Asphalt Pavements* (JTG F40—2004), the water sensibility of the DCLR-modified mixture asphalt meet the specifications under the two processing methods. The water sensibility of the DCLR-modified mixture asphalt under processing method II was lower than that of the mixture asphalt using processing method I. This meant that the water sensibility of the mixture asphalt was reduced when adding DCLR.

CONCLUSIONS

When DCLR blends with a binder, the high-temperature properties of the binder can be improved, but the low-temperature properties of the binder can be sharply degraded. In order to improve the blend’s low-temperature properties, furfural extract oils can be added into the blends.

The performance of the DCLR-modified mixture asphalt satisfied the specification requirements, especially the high and low-temperature characteristics. This indicated that the performance of mixture asphalts can be increased by adding DCLR.

The processing method impacted the performance of the DCLR-modified mixture asphalt; processing method II is suggested.

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Planning and Design for Prevention of Incorrect Turns onto Highway-Rail Grade Crossings

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ABSTRACT

In recent years, a number of injury and fatal collisions have occurred at certain highway-rail grade crossings located adjacent to highway intersections driveways or freeway on-ramps. Some guide signage, pavement markings, and other traffic control devices currently present at the crossings may have confused drivers and caused them to turn onto the railroad tracks rather than at the nearby intersections, driveways, or freeway on-ramps. Due to limited research in this new area of prevention of incorrect turns at highway-rail grade crossings, a research project was sponsored by the National Center for Transit Research and Florida Department of Transportation to seek cost-effective solutions. This paper first provides an intensive literature review to identify available sources of information and best practices throughout the world for innovative and cost-effective infrastructure treatments. It then presents analysis of the contributing causes of crashes that occurred in the past and offers a set of countermeasures for planning and design aimed at improving incorrect turns at highway-rail grade crossings.

INTRODUCTION AND BACKGROUND

A number of injuries and fatal crashes have occurred recently at certain highway-rail grade crossings that are located adjacent to highway intersections driveways or freeway on-ramps. Some guide signage, pavement markings, and other traffic control devices currently present near or at the crossings may have confused drivers and caused them to turn onto the railroad tracks rather than at the nearby intersections, driveways, or on-ramps. This safety issue is known to have occurred at a number of crossings in Florida and the U.S. In addition to confusing signs and markings, it is also believed that lack of proper illumination at night time made the crossings susceptible to being confused with a driveway, side street, or freeway entrance ramp.

Although collision frequency is low, the potential injuries, property damage, and lost time caused by minor incidents are magnified due to the passenger volumes that transit services handle. In addition, societal losses due to disruption in service are

increased. Most often, trains cannot avoid collisions with vehicles at crossings due to the extensive stopping distance required. Crashes may have the potential to affect the safety of tens to hundreds of passengers and nearby residents. An initial assessment, as shown in Figure 1, found that Hillsborough, Polk, Miami-Dade, Palm Beach, Duval, Orange, Broward, Escambia, and Bay counties in Florida account for more than 50 percent of the railroad crossings. Areas covered by high-frequency commercial rail also can be observed (e.g., Hillsborough, Polk, and Orange counties). An assessment from the Federal Railway Administration (FRA) for 2011 of crashes involving trains showed 52 crashes in Florida. After reviewing the incident narratives, it was found that more than five percent of the crashes involved either incorrect turns or vehicles trapped in slow-moving traffic lines.

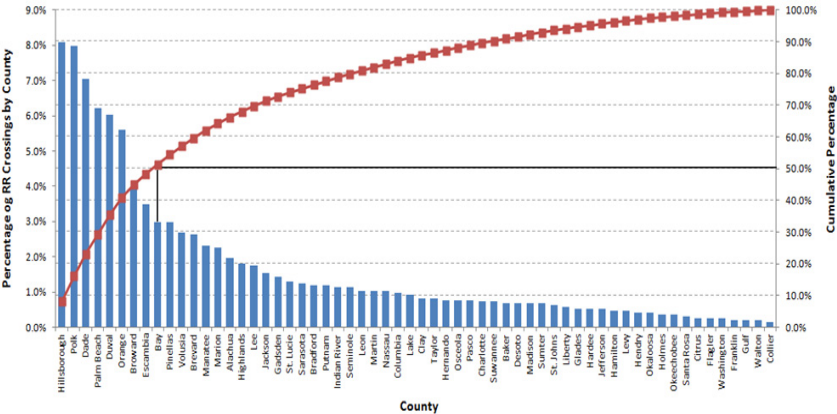


Figure 1. Railroad Crossings in Florida by County.

LITERATURE REVIEW

Events involving drivers turning incorrectly onto railroad tracks are being reported with increased frequency. This situation is observed primarily at highway-rail crossings adjacent to traffic intersections where turning movements are required (e.g., intersections, driveways, ramps, etc.). Guide signage and other traffic control devices are present in the surroundings of the crossings; however, drivers may get confused and perform incorrect turning maneuvers, leading to a potentially dangerous situation. Figure 2 presents an example of a scenario involving an incorrect turn onto railroad tracks near an on-ramp. Drivers unfamiliar with the area or who are distracted may enter the turning lane and make a wrong decision at the apparent turning point. Once the driver passes the shoulder and the lateral concrete slab, the vehicle will likely get stuck on the railroad tracks, leading to a hazardous situation. In some cases, it has been reported that drivers have turned onto railroad tracks due to GPS turning indications (Berger, 2008). A review of the narrative of FRA data from 2010 to 2012 on crashes at railroad crossings revealed one occurrence of an incorrect turn due to a GPS indication in San Francisco in 2012. More blame on wrong GPS instructions were found in Internet and newspaper articles.

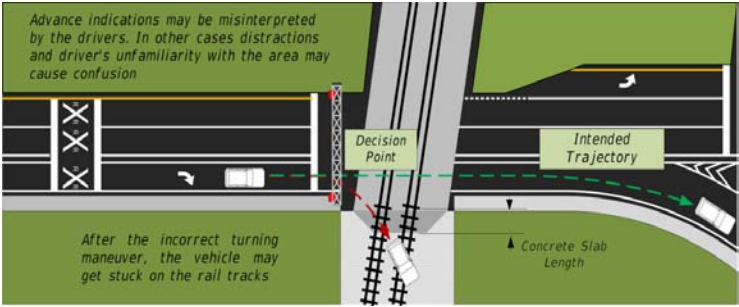


Figure 2. Sketch of Incorrect Turning Maneuver near An Interstate Ramp.

The main elements that contribute to incorrect turns onto railroad tracks generally include driver factors, traffic infrastructure factors, and environmental factors. Figure 3 presents a conceptual overview of the factors affecting incorrect turns onto railroad tracks. The framework was adapted from the behavioral framework for speeding by Campbell et al. in NCHRP Report 600, *Human Factors Guidelines for Road Systems* (Campbell, 2008). This FDOT research project focused on traffic infrastructure and environmental factors that can be modified with countermeasure elements to provide an improved driving environment on roadways adjacent to at-grade rail-highway crossings near intersections or turning points.

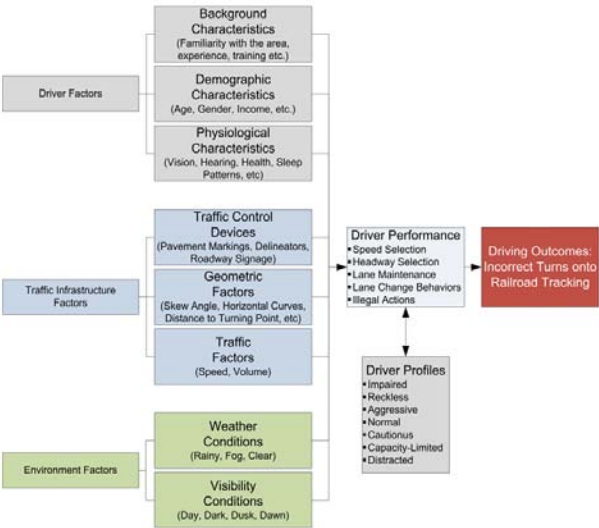


Figure 3. Factors Affecting Incorrect Turns onto Railroad Tracks.

Little data are available to produce an adequate assessment of the effect and countermeasures of driver factors. Distracted driving or driving under the influence were not considered the focus of this research; countermeasure elements addressing

specific driving actions (incorrect turns) and environmental factors (dark conditions) were the focus of this study. To address the topic, it was necessary to establish a systematic characterization approach. An intersection allows vehicles to turn onto desired destination lanes. At the same time, an intersection centerline serves as a reference to divide the rail-highway-intersection system into two halves, one of them containing the railroad tracks (nearside). The half that does not contain the railroad track is denoted as rail track farside (see Figure 4).

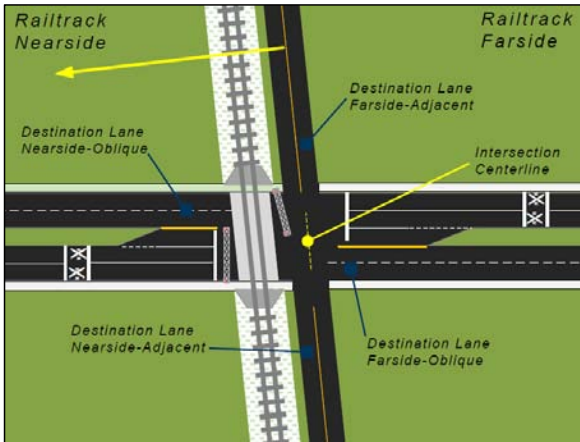


Figure 4. Naming Convention for Destination Lanes on Intersections near Railroad Crossings.

The most common intersection is the four-leg intersection containing four destination lanes. In general, it is expected that one destination lane will be immediately adjacent to the railroad track (nearside-adjacent in Figure 4). The direction of the nearside-adjacent lane may be parallel with the railroad track or may form a small skew angle. The destination lane nearside-oblique is on the rail-track side of the intersection but forms a skew angle close to 90° (above or below). The destination lane farside-adjacent may run close to parallel to the railroad track but is not immediately adjacent because it has an intersection approach lane in between. The last destination lane is the farside-oblique, which forms an angle with the railroad crossing but is located past the intersection centerline on the distant side from the rail tracks. This naming convention is used to explain the different turning scenarios.

Approach to Review of Countermeasures Based on Locations. Another important aspect of the problem of incorrect turns onto railroad tracks is the location and action of the countermeasure. Based on a systematic analysis of the turning maneuvers, the locations of the countermeasures were divided with respect to the critical point, which, in this case, is the railroad track. The recommended classification based on location is upstream, at the critical zone, and downstream of the railroad crossing, as shown in Figure 5. The critical zone is the area of the road where drivers may have a chance to correct their trajectory if provided with appropriate indications. The critical

zone may vary depending on the geometry and the scenario being treated. The countermeasures can be classified, based on the type of action, as those promoting correct path usage and those discouraging incorrect turns.

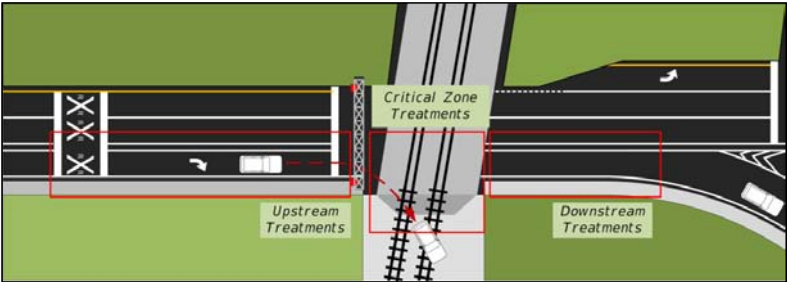


Figure 5. Countermeasure Classification Based on Location.

The countermeasures were classified with respect to upstream treatments, critical zone treatments, and downstream treatments (see Figure 6). The treatments can be further classified based on their focus. Some treatments are aimed at promoting path continuation across the railroad crossings, other treatment groups explicitly focus on discouraging incorrect turns, and a third group includes physical improvements such as geometric changes to the railroad crossing.

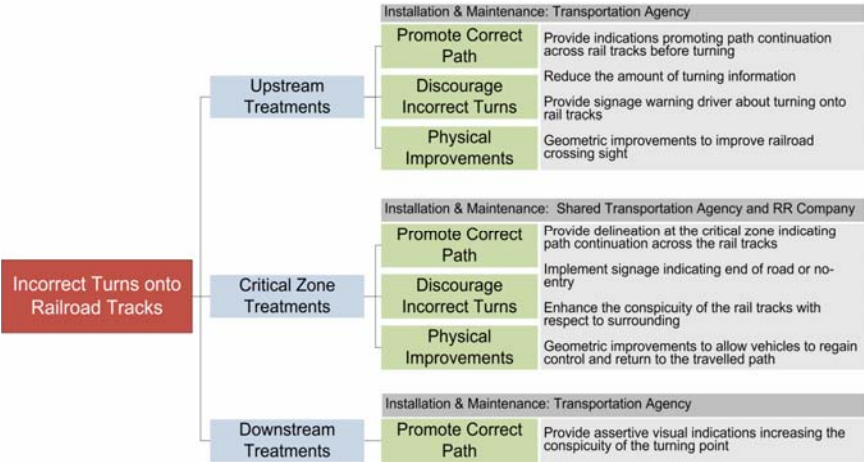


Figure 6. Taxonomy of Countermeasures to Reduce Incorrect Turns onto Railroad Tracks.

Upstream Treatments. These treatments promote the correct path, discourage incorrect turns, and make physical improvements to make the crossing more visible. They include pavement markings, striping, speed reduction, delineation devices,

vehicle activate strobe lights, active internally-illuminated signs, and advance direction signs.

Critical Zone Treatments. These treatments perform the same function as before, but they are located in the critical zone described in Figure 5. They include four-quadrant gates, sidewalk gates, side and median barriers, pavement markings, speed reduction, delineation devices, object markers, gated railway level crossing, striping, illumination, a Bodan level crossing system, pavement gate markings, and dynamic envelope pavement markings

Downstream Treatments. These treatments are generally used to promote the correct path across the railroad crossing. They include pavement markings, striping, delineation devices, and guide signs.

ANALYSIS OF CRASHES AND CONTRIBUTING CAUSES

Historical Data Compilation. The filtering process used to obtain a reduced data set of the accidents records containing incorrect turns onto rail track is presented in Figure 7. The process started with screening all the accidents from the FRA and FDOT databases from 2010–2011 and selecting only those on at-grade crossings on public roads (63% of occurrences). To focus on the objective of this study, only accidents involving passenger vehicles were selected; these accidents accounted for 31 percent of all reported accidents. After applying the intersection proximity criterion, 20 percent of the accidents remained. An additional filter to select only accidents in urban areas was included. Urban areas present more intersection density and are likely to have cases for railroad crossings near intersections and more traffic activity. Using U.S. Census spatial data, the FRA accidents were filtered by location, and only those inside urban boundaries were selected. The resulting dataset was a superset containing incorrect turn onto railroad track (ITRR) accidents. The ITRR superset contains accidents on at-grade railroad crossing involving passenger vehicles occurring in the proximity of traffic intersections in urban areas.

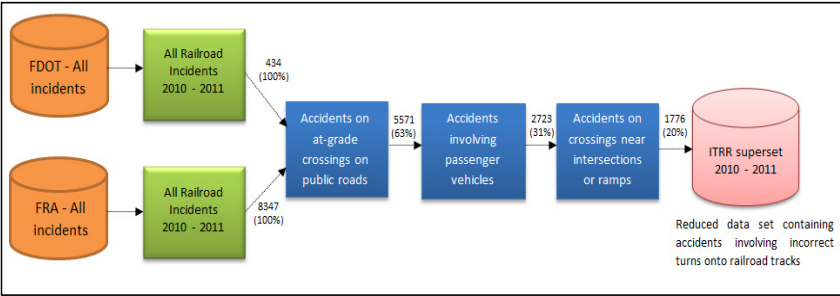


Figure 7. Data Filtering Process.

Historical Data Analysis. From the ITRR superset, the data were broken down based on injury levels, accident time of day, and weather conditions. Figure 8 presents the

results of this categorization. It can be observed that 30 percent of the accidents in the ITRR superset included fatalities and 21 percent included injuries. The distribution of the accidents based on time of day showed that 71 percent of incidents occurred during the day, 26 percent occurred at dark, and 3 percent occurred either at dusk or dawn. A total of 79 percent of the cases occurred on a clear day. To increase the likelihood of finding accidents with vehicles turning incorrectly onto a railroad track, additional filtering criteria were added to the ITRR superset. Keywords such as “stuck” and “turn” were included in the filtering criteria to narrow the search. The main assumption for this search was that railroad crossings on public roads are designed to allow safe passing for standard passenger vehicles. It is not likely that vehicles get stuck on tracks unless they veered off the road and drove onto the rail tracks. The term “turn” was included to find accidents in which it was explicitly mentioned that vehicles were turning mistakenly onto rail tracks.

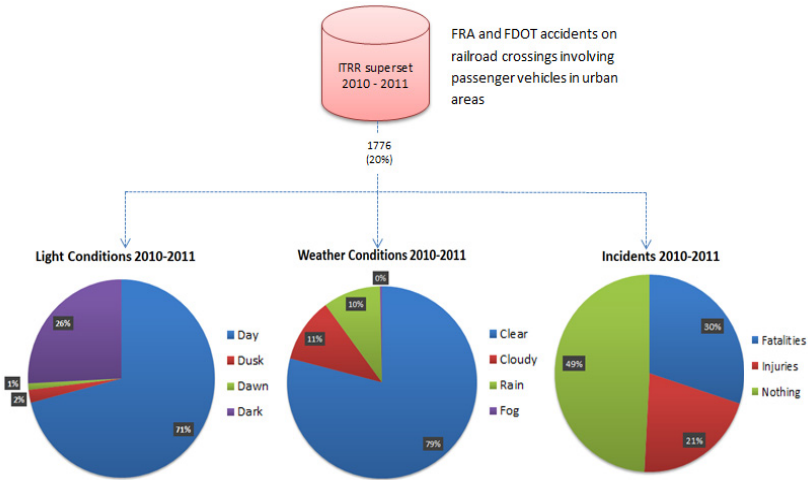


Figure 8. Correlation between Time of Day, Weather, and Occurring Incidents.

Contributing Causes. From the intensive review of the literature, historical train-vehicle crashes, and recent reported incidents of vehicles stuck at railroad tracks, as well as field observations and review of 11 selected potential candidate sites for pilot studies in Florida, the potential major causes for incorrect turns onto railroad tracks include the following:

- Confusing signs and pavement markings near highway-rail crossings
- Darkness and low visibility
- Following turn instructions from a GPS device onto railroad tracks
- Skewed at-grade highway-rail crossings
- Driver distraction

PROPOSED COUNTERMEASURES

To contribute to the mitigation of incorrect turns at highway-rail grade crossings, this study identified critical features, prevailing conditions, and potential major contributing causes influencing the incorrect turning behavior of vehicles in the proximity of a rail crossing and developed a set of countermeasures aimed at improving incorrect turns at these crossings.

The major recommended treatments for upstream of a highway-rail grade crossing include advance direction signage and striping. The recommended downstream treatments also consist of guide signs and striping. For the critical zone, treatments such as side and median barriers, illumination, striping or dynamic pavement envelope pavement markings, and pavement gate markings are recommended. Adequate illumination is essential for reducing the number of rail-vehicle crashes and stuck vehicle incidents due to incorrect turns at night. Side and median barriers would also be helpful during the night time hours with a reflective paint.

To quantify potential drivers who experience confusion or hesitation when approaching a highway-rail grade crossing, an effective methodology was developed to determine driver hesitation rates through analysis of approaching speeds at a crossing. It provided a cost-effective method to evaluate the effectiveness of any implemented treatments to prevent incorrect turns onto railroad tracks. These data will be collected and provided in a before-and after analysis in the future. In addition to speed data, illumination level data at candidate railroad crossings show the lack of adequate illumination.

SUMMARY

This research project conducted an intensive literature review; provided insight on the potential major contributing causes for incorrect turns of vehicles onto highway-rail grade crossings; recommended practical upstream, critical zone, and downstream treatments to prevent incorrect turns onto railroad tracks; and developed a simple and practical method to help quantify the effectiveness of implemented countermeasures by comparison of hesitation rates of drivers approaching a highway-rail grade crossing via speed studies before and after the implementation of selected countermeasures by agencies.

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Impacts of Rail Technology in the 21 Century U.S. Cities

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ABSTRACT

This presentation analyzes the impact of transit technologies on the urban environment and evaluates land use, station design, and transit costs as a means of projecting the best options for intermodal stations and Transit Oriented Developments. Several Commuter Rail, AMTRAK, and Freight Rail corridors may be used in the future to accommodate Heavy, Light and/or High Speed-Rail Systems in the United States, Diesel/Electric locomotives, which populate urban rail corridors, have heavy load requirements and are industrial and/or utilitarian in appearance. However, Diesel Multiple Units (DMUs) with lighter weight, increased fuel efficiency, ease of accessibility, additional capacity, enhanced performance and lower costs are competing for their market share. Currently, Monorails, People Movers, Street Cars, along with the traditional Light and Heavy Rail systems have public acceptance to enrich the urban environments surrounding the existing corridors, and enhance commuter experience. Many cities however, are at a crossroads; transitioning from an automobile culture to a rail culture, and therefore, local cities willingly embrace opportunities for better environments.

INTRODUCTION

Several Commuter, Regional, Interstate, (AMTRAK), and Freight Rail Corridors may have to accommodate heavy, light or high speed rail systems in the United States in the future. Additionally, cities are feeling the need for a change, as the U.S. automobile culture is transforming to a rail dependent and public transportation oriented culture. New technologies such as Diesel Multiple Units (DMU) may become Federal Railroad Administration (FRA) compliant and could replace the Diesel/Electric Push Pull Commuter Locomotives which have an industrial and/or utilitarian appearance. So, the question is: How can cities leverage the benefits of transit investments by enhancing stations, and thereby improving their urban environments with Transit Oriented Developments (TODs) that are conducive to growth (see Figure 1)? How can the positive features of these train technologies be modified to make stations that would propagate revitalization of neighborhoods and eliminate or reduce blight?



Figure 1. Transit Oriented Developments

EXISTING RIGHT-OF-WAY (ROW)

Railroad Right-Of-Ways (ROW) of the regional and local rail systems of the late 19th and early 20th century lie either abandoned or have extremely sparse service. Very often, these rail corridors are surrounded or paralleled by roads, highways and/or freeways (see Figure 2). Good examples of this are the San Fernando Road and the freight/commuter rail corridor in the San Fernando Valley in Los Angeles. This combined Rail-Road-Freeway corridor has made pedestrian friendly access impossible, and in some cases, even roadway access is not achieved amicably for the betterment of the environment. Often, their Civic Center and their Downtown Districts lie on one side of the tracks and freeways, while the residential neighborhoods are on the opposite side. Thus creating urban environments where face-to-face relationships between the residents is rare if not impossible, and the social scene of their town is never fully complete.

NEW ERA

Now with a new era of high speed rail development in the United States, many of the old abandoned or underutilized rail corridors provide the only opportunity to introduce high speed rail in dense urban environments. The only nucleus of hope is to enrich the urban environment around station areas which can plant the seeds of renewal and revitalization. Some city officials see this as a means of seeking private investment for revitalizing their towns, while others grapple with issues of land use, zoning and social acceptance in an automobile driven culture decades old. However, the challenge often comes from the lack of will from multiple government agencies to

collaborate and produce the state-of-the-art network of rail transit systems, and their corresponding stations.



Figure 2. Rail Corridors Paralleled by Highways

TRANSIT ORIENTED DEVELOPMENT (TOD)

TODs are successful around stations in urban areas, but this has been a long time coming. On the other hand, suburban, commuter/main line stations are often in deteriorated, industrial and blighted areas where investments are slow and often lacking. These station areas have become a cesspool of parking lots, capturing patronage from the surrounding suburbia, all in search of a city. However, tremendous opportunities exist for streamlining transit operations by introducing station standards conducive to all technologies and realigning zoning with the surroundings by improving vehicular/pedestrian access, and creating sustainable environmental investments.

Land use needs to be re-examined to allow revitalization of cities with changed zoning on height limits and building massing. Urban design studies with city models should be prepared to study the character of towns. ‘Urban linkages’ are necessary and need to be established not only for buses and automobiles, but for pedestrian walkways and bicycle corridors.

CORRECT CONDITIONS

Diesel/Electric locomotives have heavy load requirements and are industrial and utilitarian in appearance. Therefore, stations serving them fail to create an environment conducive to development. Additional noise, vibrations, dust, pollution,

and other environmental factors prevent the growth of civic and urban facilities associated with such station sites. In heavily congested and dense metropolitan environments such as those found in eastern United States, station sites invite only high use commercial and residential developments. On the other hand, cities in the western United States appear to promote highway or roadway development paralleling the track right-of-way (ROW), and any adjacent open or underutilized land, often attracts cesspools of asphalted parking lots for surrounding land uses.

The other metropolitan cities around the world have significant inter-city and inter-country passenger rail traffic, and they have customized their trains with better designs to meet passenger needs. U.S. cities are at a crossroads; transitioning from an automobile to a rail culture and therefore, there needs to be new thinking with concepts and ideas which are suitable to local city environments and social behavior.

HISTORIC CONTEXT

Sometimes cities have attempted to come up with grand ideas, but only to be thwarted by local laws, regulations and codes. A good example is the development of the Tren Urbano Project in San Juan, Puerto Rico, where the State Historic Preservation Office (SHPO) played a significant role in the development of the Rio Piedras Contract – the Underground Heavy Rail Segment running thru the heart of the Historic District of San Juan.

The Client desired a flamboyant modern urban form for the station headhouses, which were developed by our firm during the bid phases of the project. However, the State Historic Preservation Office rejected the modern concepts and then contributed immensely, and for good reason, to preserve the architectural character of this 200 year old Rio Piedras Plaza. The station headhouse design, which was right across the historic plaza and the church, was modified with façade elevations which complemented the character of the surrounding buildings and the church. (The two renderings in Figure 3 and Figure 4 show how the station design evolved and the final constructed headhouse is shown in Figure 5).



Figure 3. Rendering of How the Station Design Evolved



Figure 4. Rendering of How the Station Design Evolved



Figure 5. Final Constructed Headhouse

NEW DEVELOPMENT

Metrolink in Los Angeles, Caltrain in the Bay area (San Francisco-Oakland), TexRail in Dallas, Sounder in Seattle, and others will have future stations interfacing with High Speed Trains DMU's, Metro's, Bus Rapid Transit (BRT) and other transit forms. The federal mandate for Positive Train Control (PTC) will improve performance characteristics and mitigate collisions. The Federal Railroad Administration (FRA) waiver for specific line strength requirements will permit new rail vehicle designs with reduced axle loadings that cause less noise, vibration, and consume less energy. This scenario will create opportunities to replace Diesel Push-Pull train arrangements with DMUs, and other light and heavy rail vehicles, including street cars (see Figure 6).

Cities such as Los Angeles, New York City, and San Francisco have all felt the influence and impacts of the use of primitive rail technology. However, much has to be achieved politically and socially to improve the state of their commuter rail and/or other rail systems.



Figure 6. Future Stations Interfacing with High Speed Trains

MODERN TECHNOLOGY

The aerodynamic appearances of new technology vehicles have improved acceleration and braking potential, shortened train dwell times, and have complied with American with Disabilities Act (ADA) laws to provide the much needed accessibility for the disabled.

Cross platform transfer between high speed rail and existing commuter & regional rail systems pose many questions and demand challenging results and creative solutions. Some of the challenges are as follows:

- Platform lengths between systems
- Differing fare vending and collection operating systems
- Passenger access from both sides of existing railroad right-of-way (ROW's)
- Civic Center locations and their relationship to railroad corridors and neighborhoods

The presentation examines rail vehicle developments, potential for new station standards, and operational efficiencies that could be conducive to urban zoning changes and invite Transit Oriented Development (TOD) investments (see Figure 7).

THEME

Technology can significantly change the development aesthetics of our urban environments. However, in order to achieve the best outcome we must create and adopt new standards for seamless transfers and connections within rail systems networks, feeder bus, passenger vehicles, while allowing for access and parking.

The growth of pedestrian friendly neighborhoods with recreational, commercial, educational, civic and entertainment centers around such intermodal hubs will be the true measure of success.

NEW DEVELOPMENTS

Several new projects have sprung up all across the USA to embrace the potential of growth and development, as newer rail technology propagate Intermodal Stations and

their corresponding Transit Oriented Developments. In New York, several key projects are under development or are proposed. Some examples are as follows:

- Grand Central Terminal
- World Trade Center Station
- Fulton Street Transit Center
- Moynihan Station
- Pennsylvania Station Reconstruction

On the west coast, new energy has crept in with the development of the intermodal-multiuse Transbay Terminal in San Francisco, now under construction. The Los Angeles Union Passenger Terminal has a Master Plan in development to accommodate the newly proposed California High Speed Rail system, and several other local rail corridor development projects. Private investments are seeping in as more and more intermodal rail/road developments are being conceived and promoted by Transit Agencies. Creative architectural concepts and ideas attract citizens towards these new neighborhoods of tomorrow.



Figure 7. Potential for New Station Standards

FINAL THOUGHTS

In conclusion, the Civic Authorities and the Transit Agencies in all metropolitan cities must recognize that an investment in transit infrastructure is ultimately a civic investment for the betterment of the city as a whole, and all their transit stations are unique assets which cannot be abandoned as they belong to the citizens of the country as a whole.

Inspection Method for Track Components Defects based on Image Recognition in Subway System

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ABSTRAC

The track components such as rail, fastening, ballastless slab in urban rail transit are damaged frequently by train dynamic load or temperature and humidity of the environment around the track. Traditional inspection methods like human inspection and identification were low efficient and high missing rate. In this paper, based on the theory of Image Recognition (IR), the identification algorithm for track components condition and damages was researched. The image processing method, the identification operators and data transform method were analyzed. The image recognition method for track component damages was presented. The broken rail and crack in rail food from the photo graphics by inspection were recognized and identified. It shows this method has high accuracy and identification capability. It will be the key technology for the software of high-speed photograph technology and image recognition technology in the inspection of the track components in urban rail transit.

INSTRUCTION

Urban rail transit now plays more and more important role in people's travel as its advantage in punctuality, speed, convenient and safety. To guarantee the smoothness and comfort in transit operation, reduce the noise and the vibration, ones who take charge of the management of urban rail track should pay more attention in track components and take more strict measures to keep them from damage.

However, with the influence of increasing traffic volume, high dynamic load, long operation time, starting or accelerating & braking frequently, temperature and humidity of the environment around the track, the track components such as rail, fastening, ballastless slab in urban rail transit are damaged rapidly. Traditional inspection method like human inspection and identification cannot satisfy the demand as its low efficient and high missing rate. Ultrasonic testing performs relatively well in detecting deep surface-breaking and internal defects, while it has limitation in Rolling Contact Fatigue (RCF) and so on (Lin, 2009). As a result, a

more convenient and highly effective method is required.

Nowadays, the theory of Image Recognition (IR) and the technology of high-speed photograph are completed. IR has been applied in many areas such as platform screen door (Guo, 2009) and it achieves good results. Besides, the image processing method or identification operators provide reliable basis for real-time monitor method for track components defect. In this paper, key technologies including the identification algorithm & operators for track components condition, data transform method, image processing and recognition method, are presented and researched. According to the test result, it can be the basis for building an intelligent recognition track component defects system.

THEORIES OF INSPECTION

Theoretical basis

(1) Edge detection by the Canny operator

The purpose of Canny operator is to find the best algorithm for edge detection. To achieve it, the following aspects should be considered (Xue, 2013; Lin, 2003; Sun, 2010; Wang, 2012):

- Good detection: The algorithm can detect as more actual edge as possible.
- Good location: The marked edge by the algorithm should be close to the actual edge in maximum extent possible.
- The least response: The edge should be marked only once. At the same time, the noise of the image should be rejected.

To satisfy all above requirement, Canny operator use the variational method, which is used for searching a function to resolve special function. It is represented by the sum of four exponential function terms, while it's very similar to the first-order derivative of Gaussian function. While before using Canny operator, the noise of the image must be filtered out by the Gaussian filter (Xue, 2013; Lin, 2003; Wang, 2012). The edge of the track components such as rail, fastening and sleeper is detected by the Canny operator.

(2) Hough Line Transform

Hough Transform is widely used to identify geometry in image processing. As it is not affected by Graph Rotation, it is convenient to transform geometries. This paper, based on the simplest case of Hough transform (detecting straight lines), transforms the slope-intercept coordinate to polar coordinate in order to determine the rail outline through black and white image. That is to say, the expression of line

$y = mx + b$ is transformed to $y = \left(-\frac{\cos \theta}{\sin \theta} \right) x + \left(\frac{r}{\sin \theta} \right)$ or $r = x \cos \theta + y \sin \theta$ (Tang,

2011; Ye, 2011), where m and b are the slope and intercept respectively, θ and r are the angle and the distance of linear and origin point.

(3) Sobel operator

Sobel operator is one of the useful operators in image processing, which has strong directionality in horizontal and vertical direction (Yuan, 2009). Any pixel in

image can get its gradient vector or normal vector by using Sobel operator. It includes two groups of 3×3 matrix, representing the horizontal and the vertical one (Yuan, 2009). Convolving it with image, the approximate finite difference brightness value in horizontal and vertical direction can be gotten. In this paper, the edge in the vertical direction by a derivation of vertical direction is detected by using Sobel operator to determine the rail crack.

Implementation process

The flowchart of identifying rail crack based on Image Recognition is listed as below:

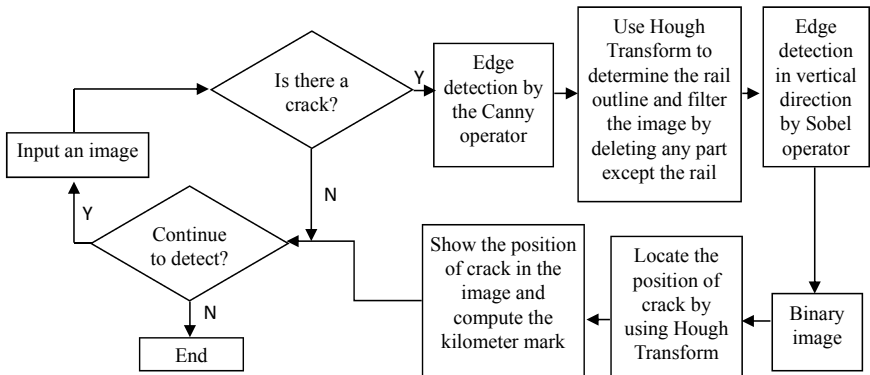


Figure 1. The Theory and Flowchart of Rail Crack Inspection

ANALYSIS

When receiving photograph, the software will show the message “No cracks found” in the image and quit if someone presses the spacebar as long as the rail is intact, see Figure 2(a):



(a) Processing intact rail



(b) Processing damaged rail

Figure 2. The Screenshot when the Software Processes the Image of Different Rail

By the time there is some crack in the image, the each step of treatment process on the image is shown by pressing the spacebar. Here in the Figure 2(b), the crack in the rail foot was artificially added just for testing. It's not the real existence.

(1) Using the Canny operator for edge detection

The software will automatically load the Canny operator for edge detection after determining the crack. The steps are listed as below:

- Noise Reduction: Filter out any noise of the image by convoluting the original image with the Gaussian filter.
- Compute the gradient and angle (Lin, 2003; Wang, 2012).

The edge in the image may point in different directions; therefore the Canny operator uses four masks to compute the edge in horizontal, the vertical and the diagonal direction respectively. The gradient strength and direction can be computed with:

$$G(x, y) = \sqrt{P_x^2(x, y) + P_y^2(x, y)}, \quad \theta(x, y) = \arctan \left[\frac{P_y(x, y)}{P_x(x, y)} \right].$$

Where $G(x, y)$ is the gradient strength; $\theta(x, y)$ is the gradient direction; $P_x(x, y)$ and $P_y(x, y)$ are the result of the convolution between the filtered image and the masks P_x and P_y . P_x and P_y are the convolution masks in x and y directions:

$$P_x = \begin{bmatrix} -0.5 & +0.5 \\ -0.5 & +0.5 \end{bmatrix}, \quad P_y = \begin{bmatrix} +0.5 & +0.5 \\ -0.5 & -0.5 \end{bmatrix}$$

- Non-maximum suppression. Non-maximum suppression is applied, which removes pixels that are not considered to be part of the edge. Hence, only edge lines will remain after this process.
- Hysteresis: Two thresholds (Kapur, 1985) (The upper and lower ratios are between 2:1 and 3:1) are applied. If a pixel gradient is higher than the upper threshold, the pixel is accepted as an edge otherwise it is rejected. If the pixel gradient is between the two thresholds, then it will be accepted only if it is connected to a pixel that is above the upper threshold. For further processing, the value of the second order directional derivative in gradient direction is defined as zero by using the formula:

$$L_x^2 L_{xx} + 2L_x L_y L_{xy} + L_y^2 L_{yy} = 0.$$

While the third order directional derivative in gradient direction should meet the conditions:

$$L_x^3 L_{xxx} + 3L_x^2 L_y L_{xxy} + 3L_x L_y^2 L_{xyy} + L_y^3 L_{yyy} < 0.$$

Variables, such as L_x , L_y ... L_{yyy} , are the partial derivative of filtered image L (http://en.wikipedia.org/wiki/Canny_edge_detector).

The edge after the above steps should be the continuous lines and there is no need to use any other method to improve the result. A binary image is produced and each pixel should be the edge point in Figure 3.

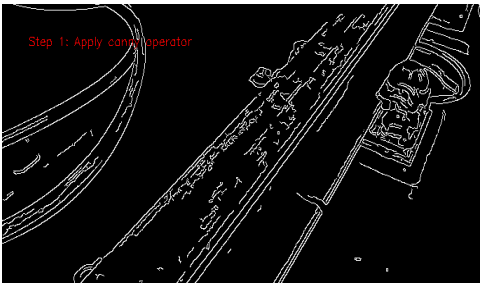


Figure 3. The Rail Image after Computing by the Canny Operator

(2) Locate the rail by Hough Line Transform

Based on the image computed by the Canny operator, Hough Transform is used to transform the Cartesian coordinate to polar coordinate for locating the position of rail with a straight line. As it's known, the point and its coordinate representing the edge has been shown, which provides a prerequisite for locating the position of the rail.

In Cartesian coordinate, for each point, there are countless lines passing it as the slope and the intercept is unknown. Each line passing the point has one number pair which is constituted by the angle and distance between coordinate origin and the line. The number pair can be represented by the polar coordinate. Similarly, any other point in the image has its number pairs. A Cartesian coordinate with angle as horizontal axis and distance as vertical axis can be built. In the coordinate, the number pairs of each point can make a curve and all of the curves that represent the points should intersect at one point in the angle-distance coordinate. The line corresponding to this point is gotten.

Take an example to explain the theory. In figure 4(a) (Hough transform, http://en.wikipedia.org/wiki/Hough_transform), point “a” is from the rail outline in figure 3 while θ is the angle and r is the distance between coordinate origin “o” and line “L”. In figure 4(b), three curves A, B, C are respectively made of number pairs corresponding to three points (point like “a” in figure 4(a)). The angle & distance of point “b” determine the line that simultaneously passes the three points. The others can follow the same method.

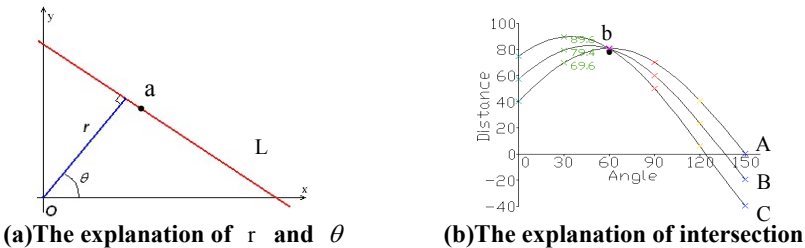


Figure 4. An Example for Explaining the Hough Transform

By determining the intersection points like “b”, the outline of rail can be shown clearly in Figure 5:

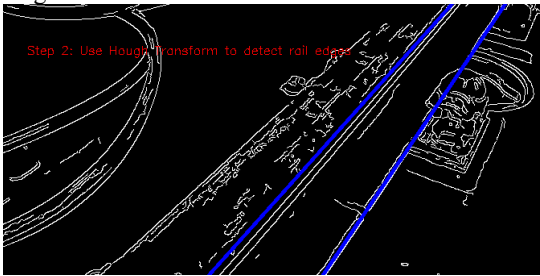


Figure 5. Define the Rail Edge by Hough Transform

(3) Get rid of clutter background

After locating the position of rail by Hough Transform, the software deletes all parts of the image except the rail to reduce the workload and the effect of other factor. That is to say, there is only rail in color is left and the other part is black. It’s shown in Figure 6.



Figure 6. Get Rid of Clutter Background of Image

(4) Detect the edge in the vertical direction by a derivation of Y-direction using Sobel operator (Yuan, 2009)

A represents the original image and G_x & G_y respectively represent the edge detection image in horizontal and vertical direction. The formula is used in the form of

$$G_x = \begin{pmatrix} -1 & 0 & 1 \\ -2 & 0 & 2 \\ -1 & 0 & 1 \end{pmatrix} * A \quad \text{And} \quad G_y = \begin{pmatrix} 1 & 2 & 1 \\ 0 & 0 & 0 \\ -1 & -2 & -1 \end{pmatrix} * A.$$

The approximate gradient and the angle of each pixel in horizontal and vertical direction can be calculated by using the formula (G is the gradient strength, θ is the gradient direction):

$$G = \sqrt{G_x^2 + G_y^2}, \quad \theta = \arctan\left(\frac{G_y}{G_x}\right).$$

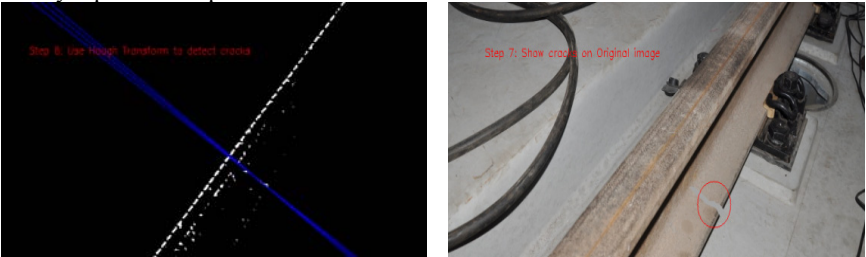
In the example, if the value of θ is zero, that means there is a vertical edge here and the left side is darker than the right one. Here, the crack is supposed mostly perpendicular to the longitudinal direction of rail for simplifying the example to find the damage or cracks that are perpendicular to the rail.



Figure 7. Sobel Operator for Crack Detection in Vertical Direction

(5) Determine the position of crack in the binary image by Hough Transform

After the Sobel operator (The crack is perpendicular to the rail), the point around the crack can be shown clearly in the binary image. Just like the principle of step (2), by using Hough Transform a series of lines can be drawn easily, which can mostly represent the position of crack. The result is as below:



(a) Use lines to mark the crack

(b) Define the crack in original image

Figure 8. Use Hough Transform to Detect Cracks

(6) Mark the crack and extract the kilometer mark for checking

When locating the crack, the system can automatically extract the kilometer mark for worker on duty. Also in this step, the Internet of Things (IOT) can be brought into the system. Information about the location of crack can be transferred to the work station and then the message can be received from the phone through 3G wireless network (Zhou, 2013). That guarantees the rail can be repaired promptly.

CONCLUSIONS

In this paper, the inspection Method for track components defects based on Image Recognition in Subway System has been researched. The theoretical basis of the algorithm has been introduced. By developing software and testing in practice, the following conclusion can be drawn:

(1) It's suitable to introduce the Image Recognition to track component system for automatically inspecting the damage or crack. Besides, through testing in practice, it can meet the requirement of maintenance and repair and the result comes to be satisfactory.

(2) The theory and algorithm has been consummate. The software runs fast and steady as well as can immediately tell whether there is crack or not.

(3) The crack-miss ratio can be reduced greatly. What' more, it can improve the efficiency and decrease the checking time and workload.

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System Design of Broken Rail Monitoring for Urban Rail Transit Based on Ultrasonic Guided Waves

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ABSTRACT

With the complex conditions in urban rail transit, the rail in continual welded rail (CWR) may be broken in some section. The track circuits in tradition signal system can find the broken rail when this damage happened. But after the state-of-the-art signal system such as Communication Based Train Control System (CBTC) was applied, the role of the track circuits disappeared. In this paper, the existing broken rail detection methods and technology were summarized. A monitoring system of broken rail for urban rail transit based on ultrasonic guided waves was proposed combined with automatic data collection and transmission technology. Considering of overall system, the level and frame structure of the system was analyzed. Moreover, the signal frequency of ultrasonic guided wave, the type and location of broken rail, and the data transmission methods were discussed. A large number of laboratory tests were carried on to determine different calibration conditions.

INSTRUCTION

Rail is the main component of urban rail transit, whose quality affects traffic safety. Since the material of rail itself, badly weld, galvanic corrosion inside the tunnel, the repeated train loads, and longitudinal forces and so on, broken rail occurs frequently, which seriously threatens to train operation safety in urban rail transit. At present, there have been a number of methods on rail break detection, such as ultrasonic rail flaw detection car and trolley, track circuit for broken rail detection, embedded rail stress sensor, optical fiber detection, and ultrasonic guided waves detection and so on (Shi et al., 2010; Chen, 2000; Contemporary China Railway Signal Editorial Board, 2007; Kalay, 2001; Steven, 1997; Lv, 2011).

Based on the analysis of all kinds of broken rail detection methods, combination of wireless or wired data transmission technology, this paper puts forward a kind of broken rail real-time monitoring system based on ultrasonic guided wave. The transmission frequency of the guided waves, broken rail

position, the types of broken rail detection, data transmission mode and the related problems are discussed. As complexity of urban rail transit operation environment, a large number of laboratory tests are carried on to determine different calibration conditions, such as identifying broken rail, processing environmental noise, monitoring length of the rail and attenuating rate of the ultrasonic signal.

BROKEN RAIL DETECTION METHODS

Currently, the broken rail detection methods are divided into two categories: ultrasonic inspection car & trolley and track circuit. The former principle is that launching high frequency ultrasonic wave pulse signal into the rail when the car is moving on it. The signal will reflect once they encounter cracks. Through the analysis of the amplitude of reflected wave and the parameters such as transmission time, the damage situation of the rail before rail break can be found (Shi et al., 2010; Chen, 2000). The latter includes conventional track circuit and imbalance rail traction return current, all is the use of rail as a subsidiary function of track circuit to detect the broken rail after rail break. According to transportation conditions and track maintenance requirements, in recent years there have been some feasible innovation methods, such as broken rail detection by stress, ultrasonic guided wave and optical fiber, etc. These are implemented based on the non-track circuit principle.

In summary, rail ultrasonic inspection car and trolley cannot achieve continuous monitoring. The theoretical knowledge and technical means of broken rail detection method based on the principle of track circuit is relatively simple, but work states are greatly influenced by the ballast bed conditions. It can produce electric corrosion on the rail, and with the application of CBTC, the function of broken rail detection attached to the original track circuit will cease to exist. The working state of broken rail detection by optical fiber and by stress is not affected by the ballast bed conditions, but it's difficult to install and maintain. The former needs to deploy fiber after the rail breaks, and the latter cannot detect the state of broken rail between sensors so that unable to realize continuous detection. The method of ultrasonic guided wave is not affected by the ballast bed conditions and easy for installation and maintenance. This kind of detection also can find the location of broken rail when rail breaking (Tian et al., 2011).

DESIGN OF BROKEN RAIL MONITORING SYSTEM

System design. The monitoring system includes two subsystems: broken rail detection by ultrasonic guided wave and data analysis and processing. After ultrasonic guided wave conduction by rail, the signals are received and automatically recorded by receiver and then converted to electrical signals in the first subsystem. Then the signals are analyzed in the second subsystem. Because the data volume is huge, some data processing methods, such as data mining,

evolutionary computation and so on are applicable, so most of the testing data can be analyzed in the field, sending the valuable data, not the entire testing data. Then transmitted to the train control center through wired or wireless network, the corresponding maintenance measures are formulated. According to the demand of overall system structure, the monitoring system structure can be divided into the following four layers (Figure 1):

- (1) Signal source layer: Converts electrical signals into the ultrasonic signal, and realize its reflection conduction in the rail.
- (2) Data record storage layer: Real-time dynamic data collection, such as signal frequency, transmission time, waveform, etc.
- (3) Data processing and transport layer: Process and analyze data, send out alarm, pass them to the remote terminal via wireless or wired transmission.
- (4) Remote control layer: Receive alarm, determine the location and types of broken rail, formulate corresponding measures, such as maintenance or ban on the train.

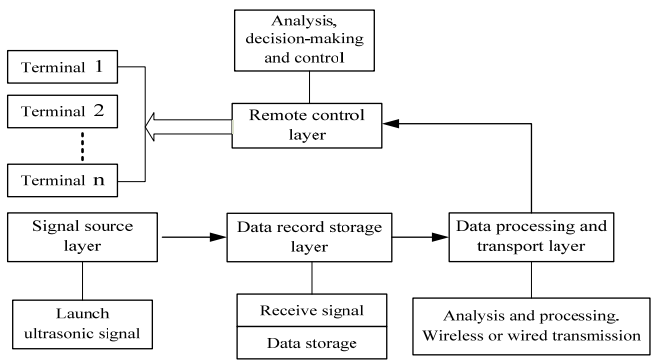


Figure 1. System Architecture.

The layout of the whole system is shown in figure 2.

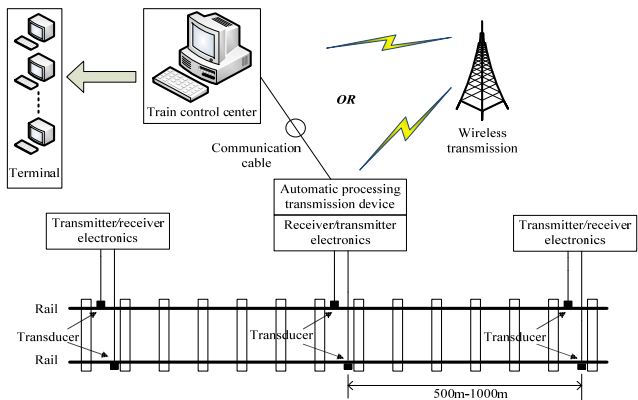


Figure 2. System Diagram.

The design of ultrasonic guided wave detection method. Ultrasonic ($f > 20\text{kHz}$) belongs to the category of mechanical wave. On a semi-infinite elastic medium surface, or at the junction of two semi-infinite elastic medium, ultrasonic will occur mode conversion for a reflection or transmission due to the discontinuity of dielectric properties. Subsequently, the various types of reflected wave and transmitted wave spread in their constant speed separately, and velocity is only related to the elastic properties of medium and material density, does not depend on the properties of the wave itself (Ren, 2010). Take Figure 3 as an example, two thick black horizontal indicate rail web on both sides, thus formed the "layer" of a certain thickness, namely thickness of the rail web. Ultrasonic wave in rail web bounces back and forth several times and propagates along the rail longitudinally. The round will produce a complex wave type conversion, and complex intervention will be happened between waveforms, namely rail web on both sides guide ultrasonic propagation in the medium. Such kind of ultrasonic wave called ultrasonic waveguide, and the ultrasonic that propagates in the waveguide is called ultrasonic guided wave (Ren, 2010).

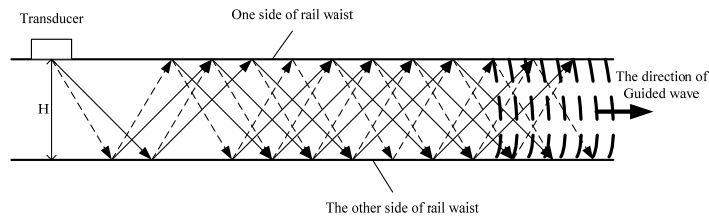


Figure 3. Ultrasonic Guided Wave Propagation inside Rail. (The Vertical View of Rail Waist)

Ultrasonic can spread in a single medium well and rail has good sound catheter features, so these conditions are the physical basis of broken rail real-time monitoring systems. Because rail boundary makes ultrasonic reflected conduction constantly in the rail, ultrasonic guided wave is formed mixing of longitudinal wave, shear wave and surface wave. Shown in Figure 4, there are two detection methods: launch from both ends and receive from the middle; launch from the middle and receive from both ends. In the Figure 4, high-voltage pulses generated by the transmitter, converted to the ultrasonic signal via the transducer and transmitted into the rail, forming ultrasonic guided wave, which is received by the receiver prob. Through checking the ultrasonic signal set is received or not in a given period of time, combining the received acoustic signal amplitude and spectrum distribution, the rail condition of break or not can determined (Zhang and Ren, 2010; Ren, 2010). Effective signal is identified based primarily on the following criteria (Burger, 2012): (1) Signal frequency; (2) Pulse width; (3) Pulse repetition interval; (4) Pulse sequence continuity.

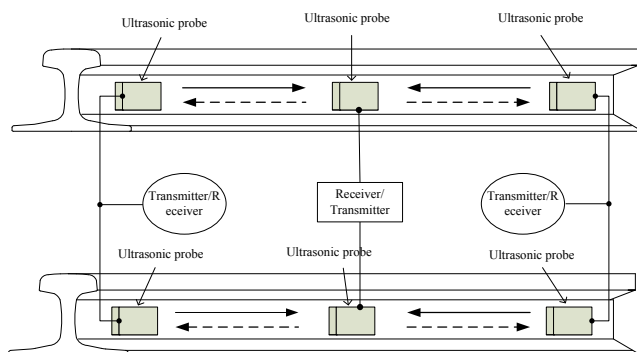


Figure 4. Structure Diagram of Broken Rail Detection.

The choice of ultrasonic guided wave detection signal frequency. Because the ultrasonic signal frequency is more than 20 kHz, its transmission in the rail is not follow the rules of one dimensional plane wave transmission. It is relatively difficult to calculate the optimal acoustic frequency used in broken rail detection through the analysis of the traditional mathematical model, so some tests have been carried on to determine the best acoustic frequency. At Pennsylvania State University, Rose et al. (2002) researched the relationship between guided wave frequency and propagation distance in California BART line; Loveday (2000) analyzed the guided wave transmitting in the rail in African Spoornet line of 2.5km. From the results they found that ultrasonic guided wave amplitude whose frequency between 31.6 kHz to 33.6 kHz was bigger, that is, this frequency range is advantageous to the transmission of sound waves. Therefore, frequency of 30kHz ~ 40kHz can be used tentatively for broken rail monitoring in urban rail transit.

The determination of broken rail location. After sending the broken rail alarm from the system, an estimate of its position should be identified in order to repair in time. Pulse-echo method was adopted to determine the position of broken rail, namely incentive in the same end with receiving signal. When rail broken occurred (signal in a preset time has not been received from the transmitters), sending a signal to the rail by the receiver. Assuming that the distance between the position of the rail broken and receiver is x , then $x = \frac{1}{2}T \times C$, where x is the distance between rail broken position and receiver; T is interval from transmitting to receiving the echo signal of the receiver; C is the speed of sound waves in the rail.

Broken rail type analysis. Rail broken usually includes rail vertical crack, horizontal crack, inclined crack and break completely. When rail breaks completely, the receiver cannot receive signals from the transmitter, one can

determine rail breaks in the corresponding segment; If rail has not completely broken, only vertical cracks happen between transmitter and receiver, the receiver can still receive part of the signal from the transmitter, one can judge whether the rail breaks or not by comparing the threshold. And rail horizontal crack blocks seldom on acoustic propagation along the rail longitudinally, also the effect of inclined crack is limited. So the ultrasonic guided wave broken rail detection method is not suitable for these two kinds of rail broken. Before the monitoring system carried out, laboratory calibration should be done according to different types of rail broken.

Transmission comparison. Data can be transmitted through wired or wireless. Wireless transmission technology which based on wireless network operation service is easy to carry out, but the signal may be interfered by the signal from mobile phone of passengers in the train. And every data transmission needs to pay a fee. Installation of cable transmission equipment in urban rail transit may be limited in terms of space, and the effect of environment and transmission wires on transmission signal requires further investigation. So in field application the data transmission technology should be chosen from the aspect of feasibility, adaptability and economy.

Other problems. This method of ultrasonic guided wave detecting broken rail has its unique advantages, but also the objective disadvantages exist, such as signal interference, the transmission distance, false alarm, etc. The vibration of train will produce big interference on ultrasonic. Under this situation, the trigger sensors at both ends of monitoring sections can stop monitoring system and make it dormant as the train approached, and reboot the system as train left. According to site condition, it is determined whether repeater is used to prolong the distance between the transmitter and receiver. When the system is warning, the false alarm can be identified by calculating the distance of "broken rail position" from the receiver. The frequency and propagation and attenuation characteristics of ultrasonic guided wave in the rail material are not very clear. So before the use of ultrasonic guided wave detection system, a large number of laboratory and field tests should be performed.

RESEARCHES HAVE BEEN DONE

Now some experiments of ultrasonic guided wave for monitoring rail broken have been carried out such as the effect of monitoring distance between two transducers, rail joint, sampling frequency (SF), rail type (ballasted or unballasted track) on the signal. Having power spectrum analysis on the collected data (The transmission frequency of ultrasonic is 35 kHz), the results appear as shown below.

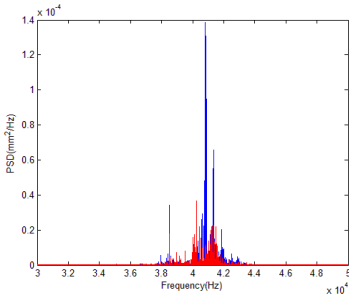


Figure 5. 0m (blue) and 25m (red) Ballasted, SF=100kHz

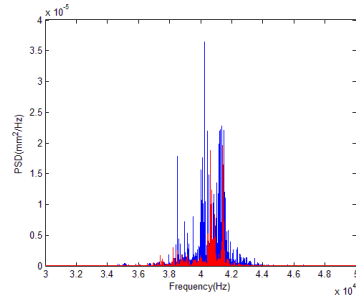


Figure 6. No joint (blue) and joint (red) Ballasted, SF=100kHz

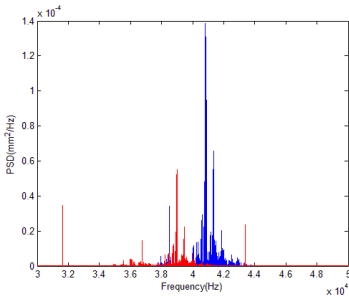


Figure 7. 100kHz (blue) and 150kHz (red), Ballasted, 0m

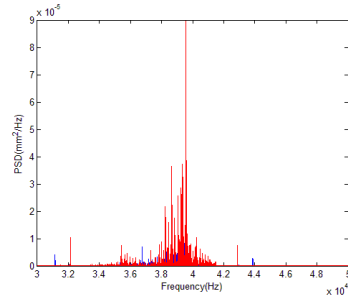


Figure 8. ballasted (blue) and unballasted (red), 150kHz, 25m

From the above diagram, it can be seen that as the distance increased or it went across the rail joint, the signal attenuated. The frequency components of originals have a decreasing trend with improving the data sampling frequency. The signal attenuation is faster propagated in the ballasted track than in unballasted track.

CONCLUSIONS

Rail broken is an urgent problem to be solved and important for guaranteeing the operation safety of urban rail transit. The broken rail monitoring system based on ultrasonic guided wave of urban rail transit is proposed to integrate the detection, data transfer and remote control, which further extends the approach and scope of rail damage research in the urban rail transit and provides a new idea for rail transit maintenance and management.

(1) Monitoring system includes four parts: signal produce, data storage, data processing, remote control;

(2) The frequency of 30 kHz ~ 40 kHz is selected preliminary. But the choose of detection method, the parameters of the effective signal, calibration of broken type and data transmission mode need a large number of indoor test;

(3) When the train goes through the monitoring section, system is suspended to a dormant state to reduce the influence of train vibration on ultrasonic;

(4) According to the actual situation one can install repeater to extend the monitoring range. When the system issues a warning, we can judge whether it is a false alarm or not by comparing the broken rail position with the length test segment.

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Evaluation and Optimization of Subway Plan Based on Complex Network

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ABSTRACT

To research the evaluation and optimization of an urban subway plan, this paper selects Changsha as a case study and designs the models of a subway network and an urban spatial network based on the subway plan and urban spatial plan of Changsha. We represent 14 districts as nodes and regard 91 connections between the districts as edges from the perspective of complex network analysis. By analyzing the node power, edge weight, and coordination degree between the two models, we determine if the subway network can match the urban space and then determine the subway plan defects and optimized strategies.

Key Words: Complex network; subway network; urban spatial network; evaluation and optimization; Changsha

INTRODUCTION

An urban subway is not only the major way to solve urban traffic jams to achieve low-carbon goals but also will have a significant effect on urban structure, land use, and economic activities. Therefore, to rationally evaluate a subway plan is very important during the current large-scale construction era in China.

Recently, to evaluate an urban subway plan, scholars in China and abroad mainly use the methods of analytic hierarchy process (AHP), principal components analysis (PCA), fuzzy comprehensive evaluation method (FCED), grey correlation analysis method (GCM), data envelopment analysis (DEA), and entropy weight analysis method (EWA). Limited by their own research backgrounds and discipline scopes, these methods usually have many defects such as experience and judgment, leading to a subjective result, and statistical data defects, thus increasing operation cost for plan comparison but not for plan optimization. Therefore, an objective, succinct and precise evaluation method

needs to be constructed.

From the view of a complex network, on the basis of a subway plan and an urban spatial structure plan, we divided Changsha into 14 districts which are taken as the nodes and whose connection strengths are the edges of a subway network and a spatial network. By comparing the two models, we analyzed the compatibility between the Changsha subway plan and the spatial plan and put forward the evaluation and the optimization of the subway plan.

NETWORK MODEL CONSTRUCTION

Subway network model. From the view of a traffic zone, we selected the statistics as follows: (1) number of transfer stations, (2) number of stations, (3) number of subway lines, (4) station density of construction land, and (5)–(11) number of stations on the shortcut from the districts to each subway line as the eleven influential elements of subway network. The first four elements are positive indexes; the last seven are reverse indexes. We transformed the statistical data to standard data using the following method:

$$y_{ij}' = \begin{cases} \frac{y_{ij} - y_{j\min}}{y_{j\max} - y_{j\min}} & y_{ij} \in \{positive\ index\} \\ \frac{y_{j\max} - y_{ij}}{y_{j\max} - y_{j\min}} & y_{ij} \in \{reverse\ index\} \end{cases}$$

where $y_{ij}' \in \{Y_S'\}$, Y_S' is the standard matrix, $y_{ij} \in \{Y_S\}$, Y_S is the original matrix, $y_{j\max}$ is the maximum value in column j of Y_S , $y_{j\min}$ is the minimum value in column j of Y_S .

Then, we transformed the incidence matrix to the adjacency matrix with the minimum method as follows:

$$x_{im} = \begin{cases} \sum \min(y_{ij}', y_{mj}') & i \neq m \\ 0 & i = m \end{cases}$$

where $x_{im} \in \{X_S\}$, $\{X_S\}$ is the original adjacency matrix, $y_{ij}', y_{mj}' \in \{Y_S'\}$.

To facilitate the comparison, we transformed the absolute data to the relative data with the linear proportional method:

$$x_{ij}' = \frac{n(n-1)x_{ij}}{\sum x_{ij}}$$

where $x_{ij}' \in \{X_S'\}$, X_S' is the relative weighted matrix, $x_{ij} \in \{X_S\}$, X_S is the absolute weighted matrix.

Table 1 shows the adjacency matrix of subway network (Matrix X_S') in Changsha.

Table 1. Adjacency Matrix of Subway Network in Changsha (Matrix X_S').

	v_1	v_2	v_3	v_4	v_5	v_6	v_7	v_8	v_9	v_{10}	v_{11}	v_{12}	v_{13}	v_{14}
v_1	0.0	0.6	1.0	1.1	0.9	0.3	1.1	1.1	1.0	1.1	1.0	0.5	0.5	0.6
v_2	0.6	0.0	0.7	0.7	0.8	0.1	0.7	0.8	0.7	0.7	0.5	0.5	0.2	0.4
v_3	1.0	0.7	0.0	1.4	1.4	0.4	1.4	1.6	1.2	1.6	1.2	0.7	0.5	0.8
v_4	1.1	0.7	1.4	0.0	1.7	0.6	1.8	1.9	1.5	1.8	1.4	0.7	0.6	1.1
v_5	0.9	0.8	1.4	1.7	0.0	0.8	1.8	2.3	1.7	2.2	1.4	0.7	0.6	1.4
v_6	0.3	0.1	0.4	0.6	0.8	0.0	0.6	0.8	0.7	0.8	0.6	0.3	0.4	0.5
v_7	1.1	0.7	1.4	1.8	1.8	0.6	0.0	2.0	1.5	1.9	1.4	0.7	0.7	1.1
v_8	1.1	0.8	1.6	1.9	2.3	0.8	2.0	0.0	1.8	2.6	1.6	0.7	0.7	1.4
v_9	1.0	0.7	1.2	1.5	1.7	0.7	1.5	1.8	0.0	1.7	1.3	0.7	0.6	1.3
v_{10}	1.1	0.7	1.6	1.8	2.2	0.8	1.9	2.6	1.7	0.0	1.6	0.7	0.6	1.4
v_{11}	1.0	0.5	1.2	1.4	1.4	0.6	1.4	1.6	1.3	1.6	0.0	0.6	0.5	0.9
v_{12}	0.5	0.5	0.7	0.7	0.7	0.3	0.7	0.7	0.7	0.7	0.6	0.0	0.2	0.4
v_{13}	0.5	0.2	0.5	0.6	0.6	0.4	0.7	0.7	0.6	0.6	0.5	0.2	0.0	0.4
v_{14}	0.6	0.4	0.8	1.1	1.4	0.5	1.1	1.4	1.3	1.4	0.9	0.4	0.4	0.0

Note: v_1 - v_{14} are the nodes of subway network.

Urban spatial network model. From the view of regional flowing space, we selected the statistics as follows: (1) population, (2) employment opportunities, (3) number of main public service facilities, (4) number of industrial zones, (5) number of CBDs, (6) number of national transportation hubs, (7) number of regional transportation hubs, (8) number of city transportation hubs, (9) number of BRT lines, (10) number of bus lanes, and (11) construction land density as the 11 influential elements of an urban spatial network. With the same steps as in subway network model, we can determine the adjacency matrix of an urban spatial network.

Table 2 shows the adjacency matrix of urban spatial network (Matrix X_U') in Changsha.

Table 2. Adjacency Matrix of Urban Spatial Network in Changsha (Matrix X_U').

	v_1	v_2	v_3	v_4	v_5	v_6	v_7	v_8	v_9	v_{10}	v_{11}	v_{12}	v_{13}	v_{14}
v_1	0.0	1.1	1.6	1.1	1.6	1.8	0.9	1.8	1.5	1.2	0.8	0.2	0.5	0.6
v_2	1.1	0.0	1.2	1.0	1.1	1.3	1.0	1.2	1.3	1.0	0.7	0.2	0.4	0.6
v_3	1.6	1.2	0.0	1.6	1.5	2.2	1.3	2.2	1.8	1.0	1.4	0.7	0.6	0.9
v_4	1.1	1.0	1.6	0.0	1.3	1.8	0.9	2.0	1.3	1.1	1.2	0.5	0.5	0.5
v_5	1.6	1.1	1.5	1.3	0.0	1.6	1.0	1.7	1.5	1.4	0.8	0.2	0.6	0.6
v_6	1.8	1.3	2.2	1.8	1.6	0.0	1.1	2.4	1.7	1.4	1.4	0.7	0.6	1.0
v_7	0.9	1.0	1.3	0.9	1.0	1.1	0.0	1.3	1.3	0.8	0.7	0.2	0.4	0.5
v_8	1.8	1.2	2.2	2.0	1.7	2.4	1.3	0.0	2.1	1.5	1.4	0.7	0.6	0.6
v_9	1.5	1.3	1.8	1.3	1.5	1.7	1.3	2.1	0.0	1.2	0.9	0.2	0.6	0.7
v_{10}	1.2	1.0	1.0	1.1	1.4	1.4	0.8	1.5	1.2	0.0	0.5	0.0	0.3	0.3

v_7	0.9	0.8	1.0	0.7	0.7	0.8							
v_8	0.8	0.8	0.8	1.0	0.9	0.5	0.8						
v_9	0.8	0.7	0.8	1.0	0.9	0.6	0.9	0.9					
v_{10}	0.9	0.8	0.8	0.8	0.8	0.7	0.6	0.7	0.8				
v_{11}	0.9	0.9	0.9	0.9	0.8	0.6	0.7	0.9	0.8	0.5			
v_{12}	0.5	0.5	1.0	0.9	0.4	0.6	0.4	0.9	0.4	0.1	0.7		
v_{13}	1.0	0.7	0.9	0.9	1.0	0.8	0.8	0.9	1.0	0.7	1.0	0.7	
v_{14}	1.0	0.8	1.0	0.6	0.6	0.7	0.6	0.6	0.7	0.4	0.8	0.5	0.8

Then we visualize the models as shown in Figure 1, Figure 2, Figure 3, and Figure 4 using the data in Table 1, Table 2, Table 4, and Table 5, respectively.



Figure 1. Districts of Changsha.

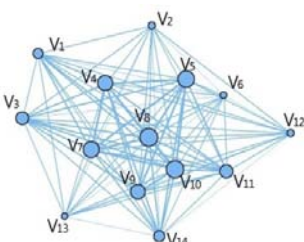


Figure 2. Subway network of Changsha.

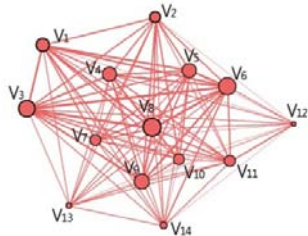


Figure 3. Urban spatial network of Changsha.

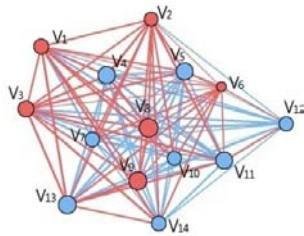


Figure 4. Coordination analysis of the two networks.

As shown in Figure 1, Changsha was divided into 14 zones. With the radius represented by node power and line width represented by edge weight, we set up the graphs of the two models, as shown in Figure 2 and Figure 3. Further, we designed a new evaluation model that represents node coordination as radius and regarding edge coordination as width. Blue indicates $S_{si} > S_{ui}$ or $w_{sij} > w_{uij}$ and red indicates the opposite situation, as shown in Figure 4.

EVALUATION AND OPTIMIZATION

Using the different colors, node radials, and line widths in Figure 4, we analyzed the coordination between the Changsha subway network and the urban spatial network. The results are shown in Table 6 and Table 7.

Table 6. Evaluation Based on Node Coordination.

Coordination	Comparison	Node
Suitable	$S_{si} \approx S_{ui}$	$v_3, v_4, v_8, v_9, v_{11}, v_{13}$
Improper	$S_{si} > S_{ui}$	v_{10}
	$S_{si} < S_{ui}$	v_2, v_6

Table 7. Evaluation Based on Edge Coordination.

Coordination	Comparison	Pair of nodes
Suitable	$w_{sij} \approx w_{uij}$	$(v_1, v_4), (v_1, v_{13}), (v_1, v_{14}), (v_3, v_5), (v_3, v_{12}), (v_3, v_{14}), (v_4, v_8), (v_4, v_9), (v_5, v_{13}), (v_9, v_{13})$
Improper	$w_{sij} > w_{uij}$	$(v_1, v_{12}), (v_2, v_{12}), (v_4, v_7), (v_4, v_{14}), (v_5, v_7), (v_5, v_{12}), (v_5, v_{14}), (v_7, v_{10}), (v_7, v_{11}), (v_7, v_{12}), (v_7, v_{14}), (v_8, v_{10}), (v_8, v_{14}), (v_9, v_{12}), (v_9, v_{14}), (v_{10}, v_{11}), (v_{10}, v_{12}), (v_{10}, v_{13}), (v_{10}, v_{14}), (v_{12}, v_{13}), (v_{12}, v_{14})$
		$(v_1, v_5), (v_1, v_6), (v_2, v_3), (v_2, v_6), (v_2, v_9), (v_2, v_{13}), (v_3, v_6), (v_4, v_6), (v_5, v_6), (v_6, v_8), (v_6, v_9), (v_6, v_{10}), (v_6, v_{11}), (v_6, v_{12}), (v_6, v_{14}), (v_{11}, v_{12})$
	$w_{sij} < w_{uij}$	

From the evaluation, we can determine that in the Changsha subway plan, Leifeng, Shifu, Zhongxin, Shengfu, Huangli, Pingpu have a rational centralization. The node power of Guitanghe is too large, and Jinxia and Xingma are small. Ten pairs of districts have a suitable edge weight, including Gaoxing-Shifu, Gaoxing-Pingpu, and Gaoxing-Muyun. The connection strength is more than that of Gaoxing-Konggang, Jinxia- Konggang, Shifu-Lushan, and 21 other pairs, while it is inadequate for Gaoxing-Xinshiji, Gaoxing-Xingma, Jinxia-Leifeng, and 16 other pairs.

To avoid the maladjustment between the subway plan and the urban spatial plan and to provide healthier development in the spatial structure, comprehensive transportation, land use, and economy activities of Changsha city, the following are suggested:

- (1) Slightly decrease the number of subway stations in Xinshiji, Lushan, Konggang, and Muyun.
- (2) Slightly increase the number of subway stations, transfer stations, and subway lines in Gaoxing.
- (3) Appropriately decrease the number of subway stations, transfer stations, and subway lines in Guitanghe.
- (4) Appropriately increase the number of subway stations, transfer stations, and subway lines in Jinxai.
- (5) Largely increase the number of subway stations, transfer stations, and subway lines in Xingma.

CONCLUSIONS

From the perspective of a complex network, this study designed models of the Changsha subway network and the urban spatial network using a measurable and visual method. By analyzing the coordination degree of the two networks, we determined the evaluation and optimization of the subway plan. This method avoids the strong subjectivity, high-cost evaluation, and elusive optimization in previous related research. It makes the subway plan well-adapted to the urban spatial plan. However, the research ignores the relationship between subways and other urban transportation, which can be improved in future study to achieve a more precise evaluation and optimization of the subway plan.

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Research on the Layout Method and Evaluation Model of High-speed Railway Stations: A Case Study of Optimizing the Layout Scheme on Xi'an-Chengdu High-speed Railway Stations

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ABSTRACT

Line planning is the primary task of high-speed railway construction; station layout design along the previously-planned line is the second phase. Optimized distribution of high-speed railway stations could improve passing capacity and operational performance of the line. First, the paper details a general layout theory of high-speed railway stations. To estimate the quantity of high-speed railway stations, a concept of “reasonable distance between stations” was introduced, which is a focused point in the planning of railway stations. The paper then establishes hierarchically an index system, measures the weights of the indexes by the Analytic Hierarchy Process, and establishes a layout evaluation model of high-speed railway stations, in which the marks of quantitative indexes are determined by membership function, and the marks of qualitative indexes are determined by fuzzy comprehensive evaluation. Finally, based on the model, the best layout scheme of Xi'an-Chengdu High-speed railway stations was determined, which was consistent with the actual situation.

LAYOUT METHOD OF HIGH-SPEED RAILWAY STATIONS

Steps.

- 1) Establish passenger stations at the distributing center cities, chokepoints, and traffic nodes, which were considered when planning the railway line.
- 2) Since city status is related to politics, economy, culture, population, administrative rank, and properties, on the condition that distance between stations is reasonable, setting up other stations should be considered in the administrative rank of the sites. Stations must be set up in capital and provincial capital cities; stations should be set up in cities in principle, but when the distance between two adjacent cities is less than 30km, whether they should be established should be determined after comparing technical economy; technical economy should also be compared before establishing stations in county-level cities or other cities below that level.
- 3) All the alternative stations on the planned line should be selected according to reasonable distance between stations.
- 4) The best layout scheme is determined by the layout evaluation model of high-speed railway stations.

Quantity of stations and optimized distance between stations. Reasonable

distance between stations is used to determine the quantity of stations, select alternative stations on the planned line, and decide on the best layout scheme. The relationship between the number of stations and the distance between stations is shown in Equation (1)

$$N = \text{int}(L/\bar{d} + 1) \tag{1}$$

Where, N = number of stations on railway line; L = length of railway line; and $\bar{d}$ = average optimal distance between stations, as shown in Equation (2).

$$\bar{d} = \max(L/n, L/K_y, \frac{I \cdot v_A \cdot v_B}{v_A - v_B}) \tag{2}$$

Where n = stop times of trains, which depends on dwell time and relaxation time of trains on railway line; K_y = times of train overtaking; and v_A, v_B = designed speed of different kind of trains.

EVALUATION MODEL BASED ON COMPREHENSIVE ANALYSIS OF QUANTITATIVE AND QUALITATIVE INDEXES

Index system. The Index System is divided into two layers, as shown in Figure 1.

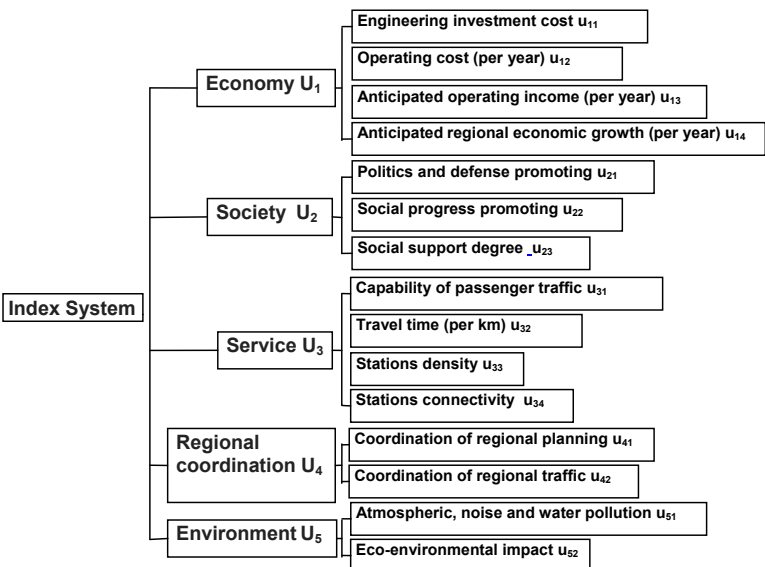


Figure 1. Evaluation indicator system of layout schemes.

Weights of indexes. Calculate the weight vector of the first grade indicators first. The judgment matrix A is derived after statistical analysis, and then the weight vector W is derived by standardizing, as shown in Table 1.

Table 1. Judgment Matrix and Weight Vector of First Grade Indexes.

A	U_1	U_2	U_3	U_4	U_5	W	$\lambda_{\max} = 5.2$
U_1	1.00	4.00	0.33	3.00	3.00	0.25	$CR = 0.05 < 0.1$
U_2	0.25	1.00	0.20	0.33	0.33	0.06	Passing consistency check
U_3	3.00	5.00	1.00	4.00	4.00	0.45	
U_4	0.33	3.03	0.25	1.00	1.00	0.12	
U_5	0.33	3.03	0.25	1.00	1.00	0.12	

The weights in the second grade can be calculated by the same method. Therefore, the results after consistency check are shown as follows:

$$w_1 = (w_{11}, w_{12}, w_{13}, w_{14}) = (0.3, 0.2, 0.25, 0.25)$$

$$w_2 = (w_{21}, w_{22}, w_{23}) = (0.4, 0.4, 0.2)$$

$$w_3 = (w_{31}, w_{32}, w_{33}, w_{34}) = (0.4, 0.3, 0.2, 0.1)$$

$$w_4 = (w_{41}, w_{42}) = (0.5, 0.5)$$

$$w_5 = (w_{51}, w_{52}) = (0.5, 0.5)$$

Evaluation method. In the index set, $U_X = (U_1, U_3)$ is quantitative index set and $U_Y = (U_2, U_4, U_5)$ is qualitative index set. The quantitative and qualitative indexes were processed separately in this paper.

Quantitative indexes. Suppose $U_i = (u_{i1}, u_{i2}, \dots, u_{ik_i}) \in U_X$, of which the weight set is $w_i = (w_{i1}, w_{i2}, \dots, w_{ik_i})$, $\sum_{j=1}^{k_i} w_{ij} = 1$, the score of the first grade index

U_i of scheme R can be expressed as follows: $f_r(U_i) = w_i \sum_{j=1}^{k_i} f_{ij}(u_{ij})$. $f_{ij}(u_{ij})$ is the score of the second grade indexes, $f_{ij}(u_{ij}) = w_{ij} \lambda_{ij}(u_{ij})$. $\lambda_{ij}(u_{ij})$ is the membership degree of the second grade indexes, which is obtained by establishing a membership function model of quantitative indexes as shown in following equations.

By using the linear dimensionless method, for the larger the better indexes, the membership function is shown in Equation (3).

$$\lambda_{ij}(u_{ij}) = c_1 + (c_2 - c_1) \frac{u_{ij} - \min\{u_i\}}{\max\{u_i\} - \min\{u_i\}} \quad (3)$$

Where, c_1, c_2 = the lower limit and upper limit.

For the smaller the better indexes, the membership function is shown in Equation (4).

$$\lambda_{ij}(u_{ij}) = c_1 + (c_2 - c_1) \frac{\max\{u_i\} - u_{ij}}{\max\{u_i\} - \min\{u_i\}} \quad (0 \leq c_1 < c_2 \leq 1) \quad (4)$$

So the value of the first grade index U_i of the scheme R should be expressed as

follows: $f_r(U_i) = \sum_{j=1}^{k_i} w_{ij} \lambda_{ij}(u_{ij})$. Finally, the total value of quantitative indexes

of scheme R is $X_r = \sum_{i=1}^n f_r(U_i)$.

Qualitative indexes. For all the qualitative indexes, we unified the comment set as follows:

$$V = \{V_1, V_2, \dots, V_k\} = \{V_1[c_1^1, c_1^2], V_2[c_2^1, c_2^2], \dots, V_k[c_k^1, c_k^2]\}$$

Experts give comment grades to every index. The evaluation matrix can be

$$R = \begin{pmatrix} r_{11} & r_{12} & \dots & r_{1k} \\ r_{21} & r_{22} & \dots & r_{2k} \\ \dots & \dots & \dots & \dots \\ r_{n1} & r_{n2} & \dots & r_{nk} \end{pmatrix}$$

obtained by statistical analysis: r_{ij} is the amount ratio of U_i being given remark V_j . Suppose $U_i = (u_{i1}, u_{i2}, \dots, u_{ik_i}) \in U_Y$, of which the weight set is $w_i = (w_{i1}, w_{i2}, \dots, w_{ik_i})$, $\sum_{j=1}^{k_i} w_{ij} = 1$. By fuzzy comprehensive

evaluation, we deduced $B_i = w_i \cdot R_i = (b_{i1}, b_{i2}, \dots, b_{ik_i})$. According to the principle of maximum membership degree, the membership degree of U_i should be $b_{is} = \max(b_{ij}), (j = 1, 2, \dots, k_i)$. The value of the first grade index U_i of scheme R should be $f_r(U_i) = w_i(c_{is}^1 + b_{is}(c_{is}^2 - c_{is}^1))$. Finally, the total value of qualitative

indexes of scheme R is $Y_r = \sum_{i=1}^n f_r(U_i)$.

COMPREHENSIVE EVALUATION OF LAYOUT SCHEMES OF XI'AN-CHENGDU HIGH-SPEED RAILWAY STATIONS

According to the theory of station layout and the distributing and engineering conditions of sites along the line, with the goal of minimizing investment, Scheme 1 is put forward; with the goal of optimizing distance between stations, Scheme 2 is put forward; and with the goal of maximizing service function, Scheme 3 is put forward.

Description of schemes.

Scheme 1: Centralized establishment. The passenger traffic volume of Xi'an, Hanzhong, Guangyuan, and Jiangyou take up more than 60% of the whole volume on the Xi'an-Chengdu high-speed railway. After synthetically analyzing the layout principle and considering all the indexes, such as passenger flow, politics, economics, etc., the four cities were determined to be those that should be set up on the line. Then, stations were determined to be set up in Huxian, Fopingxian, and Yangxian, Ningqiangxian in Shanxi and Jiangexian in Sichuan according to line alignment and reasonable distance between stations. In addition, for satisfying the need of

transportation ability and increasing the flexibility of the transportation organization of the high-speed railway line, on the condition that the distance between stations is reasonable, it was determined those two stations should be added in each of Xi'an, Guangyuan, and Jiangyou. There are 12 stations alone the line. The distance between Xi'an and Jiangyou is 510km, and the average distance between stations is 43km.

Scheme 2: Uniform establishment. To attract more passengers and contribute more to the local economy and passenger service along the line, we considered increasing stations in the cities at the county-level or above that have relatively dense population. Stations at Chengguxian and Nanzhengxian were added in Scheme 2, based on Scheme 1, to make the distance between stations more uniform. In this scheme, there are 14 stations on the line, and the average distance between stations is 34km.

Scheme 3: Separately establishment. To meet the need of passengers along the line, in addition to the 14 stations in Scheme 2, four stations should be added in Mazhao, Jinshui, Yanmen, and Xiaoxiba. There are 18 stations on the line in Scheme 3, and the average distance between stations is 27km.

Evaluation.

Quantitative indexes. In the index set, $U_x = (U_1, U_3)$ is quantitative index set, in which economic factor U_1 includes four indexes $u_{11}, u_{12}, u_{13}, u_{14}$. The weight set of U_1 is $w_1 = (0.3, 0.2, 0.25, 0.25)$. Taking engineering investment cost as an example, the investment costs of the three schemes are 640billion, 647billion, and 663billion. u_{11} and u_{12} are the larger the better indexes. Taking $c_1 = 0.85, c_2 = 0.87$, according to formula (4), the values of $\lambda_{11}(u_{11})$ in the three schemes are 0.880, 0.864, and 0.850. Taking $c_1 = 0.80, c_2 = 0.82$, the values of $\lambda_{12}(u_{12})$ in the three schemes are 0.820, 0.814, and 0.800. Similarly, u_{13} and u_{14} are the smaller the better indexes. Taking $c_1 = 0.80, c_2 = 0.82$, according to formula (3), the values of $\lambda_{13}(u_{13})$ in the three schemes are 0.800, 0.810, and 0.820, and the values of $\lambda_{14}(u_{14})$ in the three schemes are 0.800, 0.807, and 0.820. We therefore obtain:

$$f_1(U_1) = 0.25 \times (0.3 \times 0.88 + 0.2 \times 0.82 + 0.25 \times 0.8 + 0.25 \times 0.8) = 0.207$$

$$f_2(U_1) = 0.25 \times (0.3 \times 0.864 + 0.2 \times 0.814 + 0.25 \times 0.81 + 0.25 \times 0.807) = 0.207$$

$$f_3(U_1) = 0.25 \times (0.3 \times 0.85 + 0.2 \times 0.8 + 0.25 \times 0.82 + 0.25 \times 0.82) = 0.206$$

As illustrated in the example above, we can obtain:

$$f_1(U_3) = 0.45 \times (0.4 \times 0.85 + 0.3 \times 0.8 + 0.2 \times 0.8 + 0.1 \times 0.83) = 0.370$$

$$f_2(U_3) = 0.45 \times (0.4 \times 0.861 + 0.3 \times 0.811 + 0.2 \times 0.813 + 0.1 \times 0.820) = 0.375$$

$$f_3(U_3) = 0.45 \times (0.4 \times 0.88 + 0.3 \times 0.82 + 0.2 \times 0.82 + 0.1 \times 0.8) = 0.379$$

Finally, Scheme 1: $Y_1 = f_1(U_1) + f_1(U_3) = 0.207 + 0.370 = 0.577$; Scheme 2: $Y_2 = f_1(U_1) + f_1(U_3) = 0.207 + 0.375 = 0.582$; and Scheme 3: $Y_3 = f_1(U_1) + f_1(U_3) = 0.206 + 0.379 = 0.585$.

Qualitative indexes. In index set, $U_y = (U_2, U_4, U_5)$ are qualitative indexes. Taking the same grade comment for all qualitative indexes as following: $v =$

{excellent (0.9, 1), good (0.8, 0.9), medium (0.7, 0.8), qualified (0.6, 0.7), poor (0.5, 0.6)}. Taking social factor as an example, it includes three indexes: $U_2 = (u_{21}, u_{22}, u_{23})$, of which the weight set is $w_2 = (0.4, 0.4, 0.2)$, the matrix can be obtained by adding up the evaluation results of 10 experts:

$$\text{Scheme 1:} \quad R_2 = \begin{pmatrix} 0.5 & 0.3 & 0.2 & 0 & 0 \\ 0.4 & 0.3 & 0.3 & 0 & 0 \\ 0.5 & 0.2 & 0.3 & 0 & 0 \end{pmatrix}$$

Thus, $B_2 = w_2 \cdot R_2 = (0.46, 0.28, 0.26, 0, 0)$. The score of U_2 in Scheme 1 is $f_1(U_2) = 0.06 \times [0.46(1 - 0.9) + 0.9] = 0.0568$.

$$\text{Scheme 2:} \quad R_2 = \begin{pmatrix} 0.6 & 0.2 & 0.2 & 0 & 0 \\ 0.4 & 0.4 & 0.2 & 0 & 0 \\ 0.5 & 0.3 & 0.2 & 0 & 0 \end{pmatrix}$$

Thus, $B_2 = w_2 \cdot R_2 = (0.5, 0.3, 0.2, 0, 0)$. The score of U_2 in Scheme 2 is $f_2(U_2) = 0.06 \times [0.5(1 - 0.9) + 0.9] = 0.0570$.

Similarly, Scheme 3: $B_2 = w_2 \cdot R_2 = (0.54, 0.3, 0.16, 0, 0)$. The score of U_2 in Scheme 3 is $f_3(U_2) = 0.06 \times [0.54(1 - 0.9) + 0.9] = 0.0572$. In the same way, the scores of the other qualitative indexes U_4, U_5 can be obtained:

$$\text{Scheme 1: } B_4 = w_4 \cdot R_4 = (0.2, 0.4, 0.4, 0, 0)$$

$$f_1(U_4) = 0.12 \times [0.4(0.9 - 0.8) + 0.8] = 0.101$$

$$B_5 = w_5 \cdot R_5 = (0.1, 0.4, 0.4, 0.1, 0)$$

$$f_1(U_5) = 0.12 \times [0.4(0.9 - 0.8) + 0.8] = 0.101$$

$$\text{Scheme 2: } B_4 = w_4 \cdot R_4 = (0.3, 0.6, 0.1, 0, 0)$$

$$f_2(U_4) = 0.12 \times [0.6(0.9 - 0.8) + 0.8] = 0.103$$

$$B_5 = w_5 \cdot R_5 = (0, 0.4, 0.4, 0.2, 0)$$

$$f_2(U_5) = 0.12 \times [0.4(0.9 - 0.8) + 0.8] = 0.101$$

$$\text{Scheme 3: } B_4 = w_4 \cdot R_4 = (0.4, 0.4, 0.2, 0)$$

$$f_1(U_4) = 0.12 \times [0.7(0.9 - 0.8) + 0.8] = 0.104$$

$$B_5 = w_5 \cdot R_5 = (0, 0.3, 0.4, 0.2, 0.1)$$

$$f_2(U_5) = 0.12 \times [0.4(0.8 - 0.7) + 0.7] = 0.0888$$

Finally, Scheme 1: $Y_1 = f_1(U_2) + f_1(U_4) + f_1(U_5) = 0.0568 + 0.101 + 0.101 = 0.259$;

Scheme 2: $Y_2 = f_2(U_2) + f_2(U_4) + f_2(U_5) = 0.0570 + 0.103 + 0.101 = 0.261$; and Scheme

3: $Y_3 = f_3(U_2) + f_3(U_4) + f_3(U_5) = 0.0572 + 0.104 + 0.0888 = 0.250$.

Final comparison and selection. The total evaluation scores of the quantitative and qualitative index sets in the three schemes are as follows:

$$\text{Scheme 1: } M_1 = X_2 + Y_2 = 0.577 + 0.259 = 0.836.$$

Scheme 2: $M_2 = X_2 + Y_2 = 0.582 + 0.261 = 0.843$.

Scheme 3: $M_3 = X_3 + Y_3 = 0.585 + 0.250 = 0.835$.

Scheme 2 is the optimal scheme.

BRIEF SUMMARY

The number of stations of the three schemes is close to each other, so the total evaluation scores are similar. On the condition that the demand of transportation ability and transportation organization is satisfied, the number of stations set up in Scheme 1 is the least. At the same time, the project investment, operational expenditures, and average travel time in Scheme 1 are the least as well. However, the service function in Scheme 1 is weaker than the others. In Scheme 3, the density of stations is large, which can attract more passengers and match up better with city planning, but the larger number of stations reduces the speed advantage of high-speed rail and increases the cost. In the meantime, the economic growth it brings is limited. In Scheme 2, the number of stations is appropriate, the distance between stations is reasonable, the service function stands out, and income and expenditure are balanced. To maximize the benefit, the second scheme is recommended, which corresponding with the actual situation.

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Study on Guidance Efficiency of Pedestrian Sign in Integrated Rail Transit Hub

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ABSTRACT

Pedestrian speed is a key factor in pedestrian evacuation analysis. The speed is closely related to the guidance efficiency of pedestrian sign system. The guidance efficiency of pedestrian sign system was defined in this paper. A test method for the extraction of the guidance efficiency index was also confirmed through the pedestrian test in integrated rail transit hub. The level of the guidance efficiency of pedestrian sign system was also evaluated preliminarily in the paper. The guidance efficiency not only can be used to evaluate the guidance performance of the pedestrian traffic oriented sign in the transport hub but also can correct the pedestrian speed which is the key factor in pedestrian evacuation analysis. The difference of the evacuation time in each period between the original evacuation model and real evacuation situation can be clearly demonstrated. Therefore, the results of the pedestrian evacuation analysis will be more reliable based on this study.

INTRODUCTION

The pedestrian sign system is designed to allow people arrive at the destination fast and safely. But when people are in the space of the underground stations, they lack external reference to identify the direction, so the importance of the pedestrian sign system is prominent under these circumstances. It has been shown that at the transport hub facilities such as entrances, landmarks, airports, and stations, nearly 90 percent of the respondents think that the pedestrian sign is the most important source of information. Therefore, the level of guidance efficiency of pedestrian sign plays an important role in fulfilling its functions in rail transportation. Higher guidance efficiency can maximize the role of urban rail transportation.

Pedestrian speed is a key factor in pedestrian evacuation analysis, and the speed is closely related to the guidance efficiency of pedestrian sign system. Current research is not perfect in this respect. There is no quantitative evaluation index, no systematic analysis and evaluation system. In this case, an accurate evaluation model cannot be built without considering the influence of pedestrian sign on pedestrian speed. Finally, the resulting evacuation time would be inaccurate. In order to solve those problems, the following test methods were carried out. The definition of guidance efficiency of pedestrian sign is proposed, which is taken as an evaluation index to conduct the guidance efficiency analysis and establish the evacuation model.

GUIDANCE EFFICIENCY OF PEDESTRIAN TRAFFIC ORIENTED SIGN

The definition of the guidance efficiency of the pedestrian sign is proposed in this paper. When the Pedestrians completely rely on the pedestrian sign, the passage speed is v' , when pedestrians are completely familiar with the route and do not need the pedestrian sign, the passage speed is v , the ratio between the two is $z_s=v'/v$, so-called guidance efficiency of pedestrian traffic oriented sign.

The letter z in the index z_s represents the first letter in the name of the first author, and the letter s represents the first letter in the name of the second author. Moreover z_s is also the short of the word guidance in Chinese. So z_s is used to represent the index of guidance efficiency in this paper.

TEST METHOD

In view of the operability of the test, 29 young university students and 53 construction workers were selected as the test personnel. Test locations are arranged in a complex traffic zone underground that include several intersection and transfer hall in Beijing.

Before the beginning of the test, a pass line should be set in the test site and it should be ensured that all the testing personnel are completely unfamiliar with the test environment. It also should be set numbers of task point from the starting point of the route to the end and named them a, b, c, d and so on (see Figure 1). The section between every two task point should only include one or two kinds of pedestrian traffic oriented sign and several recorders should be arranged to record the arriving time of the test personnel at every task point. This test consists of two stages. Pedestrians' passing in a completely new station environment is simulated in the first stage. Pedestrians' normal walking speed is simulated in the second stage as the reference group.

The setting of the pass line in this test is as follows: The first part of the line is located in the subway station, it is subway station kiosk for supervision (a) → underground escalator entrance (b) → subway station south hall exports (c). The Second part of the line is located in the train station, it is subway station south hall exports (c) → first floor escalator entrance (d) → waiting hall security checkpoint (e) → 16-17 (f) wicket.

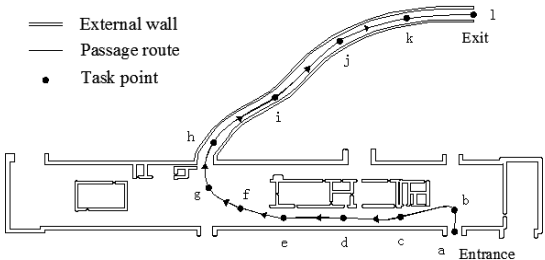


Figure 1. Sketch map of the passage route.

A. The first stage of the test

Before the first stage, it should be ensure that all the test personnel in the test environment are completely unfamiliar with the route. They pass entirely rely on the guidance of pedestrian sign. The first test personnel enter the route from a point and reach the end along b, c, d, e, and so on. The recorders then record the arriving time of every task point. When the first test personnel arrive at the end point, the second one set out. The test should be repeated 43 times. The time during which the first test personnel passed through the section ab is denoted by $t'_{1,ab}$. The length of the section is denoted by S_{ab} . Others can be obtained in a similar way. The kind of pedestrian traffic oriented sign included in each section should also be recorded.

B. The second stage of the test

Before the second stage, it should be ensured that all the test personnel in the test environment are completely familiar with the route. Now, they can finish the route without the guidance of pedestrian traffic oriented sign. Other steps are same with the first stage. The time during which the first test personnel passes through the section ab is denoted by $t_{1,ab}$. Others can be obtained in a similar way.

TEST RESULTS AND ANALYSIS

Through the observation of the pedestrian wayfinding in the test, it can be known that what influenced the pedestrian most is the basic signs system that consists of two or three kinds of signs. So the object of this paper is the signs that play a key role in the pedestrian wayfinding. As the route in the section de is so simple that pedestrian can passed glidingly without signs, so the signs in this section are out of consideration.

A. Establishment of the guidance efficiency index

The guidance efficiency index is established as follows.

The average speed of the first test personnel passing through each section in the first stage can be obtained by the following formula. Denoted them by $v'_{1,ab}$, $v'_{1,bc}$ and so on.

$$v'_{1,ab} = S_{ab} / t'_{1,ab}$$

The average speed of first test personnel passing through each section in the second stage can be obtained by the following formula. Denoted them by $v_{1,ab}$ and so on.

$$v_{1,ab} = S_{ab} / t_{1,ab}$$

Establishing the guidance efficiency of pedestrian sign of the first test personnel $z_{1,sab}$, $z_{1,sbc}$ and so on by the following formula:

$$z_{1,sab} = (v'_{1,ab} / v_{1,ab}) \times 100\%$$

Calculate the average of all the test personnel's guidance efficiency, and the guidance efficiency of pedestrian sign of each section can be obtained (see Table 1 and 2).

Table 1. Guidance efficiency of university students

Zs	section ab	section bc	section cd	section ef
average	67.7%	63.0%	65.2%	73.3%
maximum	99.4%	98.0%	93.1%	98.0%
minimum	35.4%	16.8%	18.5%	17.9%

Table 2. Guidance efficiency of construction workers

Zs	section ab	section bc	section cd	section ef
average	71.9%	60.6%	63.8%	63.9%
maximum	96.3%	98.6%	98.2%	94.9%
minimum	30.6%	22.7%	23.1%	17.2%

The way finding ability of the all test personnel have huge difference because of individual differences (speed of response, legibility distance, sense of direction and so on). Because of this, the index obtained in this test shows a certain degree of discreteness. But it can be known from the Figure 2 and Figure 3 that most of the guidance efficiency index distributed around the average and only a very small part is significantly higher or lower than the average. Therefore, the discreteness of test data exactly simulated the real pedestrian wayfinding process and it would not affect the accuracy of the guidance efficiency.

As two different types of people were selected as the test personnel, the difference of the guidance efficiency between the two groups of people can be obtained from this test. Comparing Figure 2 and Figure 3, it can be found that university students group generally has a higher level of culture, so they have better reading ability of the pedestrian signs. While, the group of construction workers in the test generally has low level of education, and some of them even cannot be read, so they have limited acceptance of the signs in the integrated rail transit hub. Based on this, the guidance efficiency of the university students group concentrated on the interval of 60%~80% and 80%~100%, when the other group concentrated on the interval of 60%~80%.

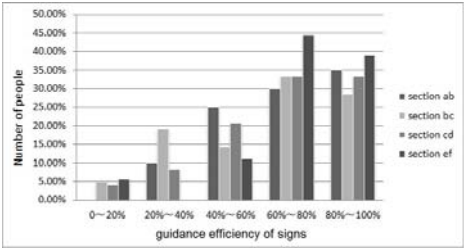


Figure 2. The number of people that distribution in each guidance efficiency level of university students group

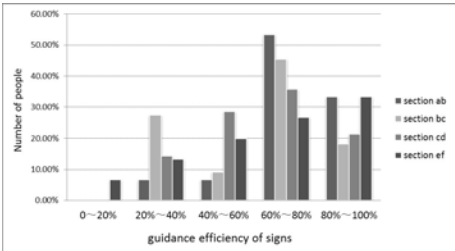


Figure 3. The number of people that distribution in each guidance efficiency level of construction workers group

B. Evaluation of the guidance efficiency index

By analyzing the test data that in this test, it can be known that the guidance efficiency of the signs in the four sections in this test are mainly distributed at the range of 60% to 80%, so the evaluation of the guidance efficiency should mainly concentrate on this range. Applying the aliquots principle, the guidance efficiency of the signs is divided into four levels: very low, low, high and very high.

When $z_s \leq 55\%$, the pedestrian speed is significantly lower than the normal walking speed [9], indicating that it could have happened several times of stopping during the pedestrian wayfinding process. So the guidance efficiency of signs is very low in this situation. When $55\% < z_s \leq 70\%$, there is still a certain gap between the pedestrian speed and the normal walking speed, indicating that it could have happened a certain times of stopping during the pedestrian wayfinding process. So the guidance efficiency of signs is low in this situation. When $70\% < z_s \leq 85\%$, the pedestrian speed is fairly close to the normal walking speed, indicating that it could have happened a few times of stopping during the pedestrian wayfinding process. So the guidance efficiency of signs is high in this situation. When $z_s > 85\%$, the pedestrian speed is very close to the normal walking speed, indicating that pedestrians can pass incessantly. So the guidance efficiency of signs is very high in this situation.

According to the evaluation principle above, the results of evaluation and classification of the pedestrian signs in the four sections in this hub is as follows (see Table 3 and Table 4).

Table 3. The guidance efficiency level of the university students group

section	ab	bc	cd	ef
level	low	low	high	high

Table 4. The guidance efficiency level of construction workers group

section	ab	bc	cd	ef
level	low	low	low	high

THE OPTIMIZATION OF EVACUATION MODEL IN THE HUB

Nowadays, studies on pedestrian evacuation in comprehensive rail transit hub mostly rely on the computer simulation. The value of pedestrian speed plays a decisive role to the simulated result in the progress of setting up pedestrian evacuation simulation model. The present accessory methods of pedestrian speed include pedestrian speed investigation and referring to the recommended value. These two methods have apparent defects that ignoring the pedestrian evacuation process sign system's influence on the speed of pedestrians, though they can get the pedestrian speed in the evacuation model. And they can't get quantitative description of the speed reduction caused by pedestrians' wrong reading of marks in the process of evacuation. Therefore, evacuation models set up on the two methods of pedestrian speed values above have certain differences with real evacuation situation, which will reduce the simulation accuracy and the evacuation time will also have certain error.

This paper defined the pedestrian sign indicated efficiency index α basing on

researches above. And pedestrian speed correction coefficient was proposed that describes the reduction degree of pedestrians speed in the process of reading marks. It described accurate quantitative of marking system's influence on the speed of pedestrians in the hub. The computation formula is as follows:

$$\alpha = \sum z_{s,basic}^i / N \quad (i=1,2,3\dots,N)$$

In which: N— total numbers of certain pedestrians;

$z_{s,basic}^i$ — Basic pedestrian indicated efficiency index of certain pedestrians.

The original speed of the pedestrians can be optimized as follows:

$$v_{cor}^i = v_{ori}^i \times \alpha_i$$

In which: v_{cor}^i — the original speed of certain pedestrians;

v_{ori}^i — The optimized speed of certain pedestrians.

Above all, v_{cor}^i is the optimized speed of all kinds of pedestrians in evacuation model that can be implied in pedestrian evacuation simulation of the hub.

The high layer evacuation model of Beijing south railway station was established and the pedestrian speed in the model was modified by this method. The correction coefficient of all kinds of people and the pedestrians speed before and after optimization are as the table below.

Table 5. Speed correction coefficient of all kinds of people in evacuation model

Category	Young and middle-aged men	Young and middle-aged women	Children	The old
α	0.716	0.716	0.716	0.535

Table 6. Speed before and after optimization of all kinds of people

Category	The original speed (m/s)	The optimized speed (m/s)
Young and middle-aged men	1.1735	0.8402
Young and middle-aged women	1.1172	0.7999
Children	0.9726	0.6964
The old	0.9849	0.5269

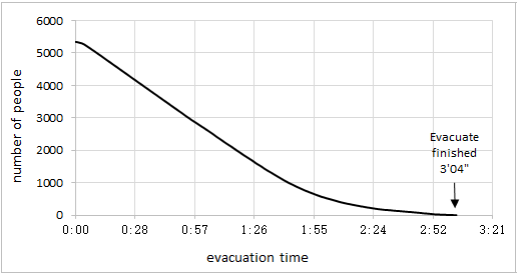


Figure 4. The relationship between the evacuation time and number of people in the model before optimized

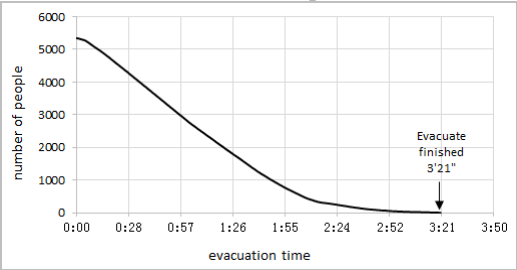


Figure 5. The relationship between the evacuation time and number of people in the model after optimized

It can be known by compared the evacuation time from the optimized model and not one that in the first stage in evacuation, pedestrians distributed in the hub need to concentrate in the evacuation exit (see Figure 4 and 5). When take no account of marks' influence, pedestrians can quickly reach the near exit and complete evacuation. While when taking account of marks' influence, pedestrian speed was reduced because of reading marks. So the time of reaching the exit is relatively longer. The different of evacuation time between the evacuation models before and after optimized was mainly produced at this stage. When all people in the hub arrived at the evacuation exit, pedestrian evacuation speed is determined by the factors such as exit width and evacuation order. At that time, the pedestrian signs had no influenced on the pedestrians. Two minutes after the start of the evacuation, most people have reached the evacuation exit. At this point, pedestrian speed before and after optimized was very close.

CONCLUSIONS

The guidance efficiency of pedestrian sign system was defined in this paper. And the guidance efficiency of signs was obtained through pedestrian wayfinding test in an integrated rail transit hub in Beijing. Different from the other qualitative research before, the proposed method is more scientific and clear, and it is worthy of the further study about pedestrian sign. The pedestrian speed obtained in the test can be used to optimize the pedestrian parameter in evacuation analysis and to make the evacuation model closer to the real situation.

Normally, the path finding of pedestrian can be divided into three types. They

are ordinary situation, leisurely situation and emergency situation. Pedestrians always find the way with an explicit destination under the ordinary situation. But they used to focus on the factors such as environmental and safety under the leisure and emergency situations. So the ordinary situation which pedestrians can observe the pedestrian sign calmly is the most representative for simulating the typical pedestrian passage. On the other hand, other two situations are also worth considering. However, these two situations are difficult to simulate and the test data are hard to be obtained directly. So the test is performed under the ordinary situation this time.

In addition, pedestrian sign system can directly affect the pedestrian evacuation time. Correcting the pedestrian speed in the evacuation model by the pedestrian speed correction factor and considering the effect of the pedestrian sign in the process of evacuation simulation are very necessary. Ignoring the effect of the pedestrian signs will lead to the less evacuation time. And it will make contribute to doing inaccurate assessment of the evaluation ability of the hub for the related research or management personnel. Furthermore, it will influence making the evacuation decision and building the auxiliary facilities of the evacuation.

ACKNOWLEDGMENT

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Studies on the Mechanical Model of Subway Station Structures under the Influence of Changes of Groundwater Level

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ABSTRACT

Based on the engineering background of Beijing urban groundwater levels rebounded, the stress behavior of subway station structure under cases of the groundwater in different water levels from the view of numerical mechanical models is researched in this paper and the working condition of this paper is the actual engineering of the Beijing subway line 10. The results demonstrate different underground water levels could lead to obvious vertical displacements and vertical stress changes of structures and the groundwater level upturn obviously within a certain range make the underground structures deform and float overall, indeed, could cause craze, water seepage, harmful ion erosion and a series of problems resulting in influencing the durability of structures. Some conclusions are important references for the further improvements of the design and construction of underground structures, reinforcement of existing substructures.

KEYWORDS

Underground water levels, underground structures, mechanical models, numerical simulation, displacement and deformation

INTRODUCTION

With the development of urban subway and complexation of underground space layout, more and more construct methods such as the new tunnel under pass the exist tunnel vertical or oblique and tunnels build in close and parallel. In Beijing the capital of china, there are many engineering example that new build tunnels under pass the exist tunnels, follow which occurs many technique problems to be solved. The research and theory on underground tunnels including relative computational model, computational method, constitutive relation and boundary condition and so on is relative hysteresis, some problems even have no rational solutions because of the complexity of geological and soil layer condition. Along with the signify effects of divert water from the south to the north project to the water source pattern of Beijing, the results maybe the water will be rebound and form new under water environment (Luo, 2011). According to the cases of civil and foreign (Simpson et al., 1989; Hurst and

Wilkinson, 1986; Brassington, 1990), some underground structure maybe crack, water seepage, floating and structure corrosion when the underground water rise (Zhang et al., 2009). For example, after reduced the exploitation of underground water, the level of water raised 50cm, leading to the subway cannot operation and the station occurred serious water seepage. In Beijing, the water level raised many times, such as underground water raised in many different districts between 1988 and 1992 because of the reduce of water exploitation in 1985 (Sun et al., 2006; Yu, 2004). However, it is difficult to analysis and evaluate the privation and control measures of engineering due to the leak of information.

This paper bases on the practice engineering of Beijing subway line7 underpass subway line 10, using the software of FLAC^{3D}, researches the mechanical behavior of underground water to existed subway station structure and shows the changes of strain and stress in different water level.

UNDERGROUND THEORETICAL ANALYSIS

The procedures of dual role in FLAC^{3D} formulaic diffusion deformation are in the skeleton of the quasi-static Biot theory and the diffusion deformation can involve single-phase Darcy flow in porous media. The different type of fluid, including gas and water, can be represented in this model. Involving variable of fluid flow in porous media is pore pressure, saturation and unit direction flow. These variables, fluid mass balance equation and Darcy liquid migration law develop basic equations of fluid responsiveness changes in pore pressure, saturation, volumetric strain and temperature. Darcy's law describes the migration of liquid. For homogeneous isotropic model, solid and liquid density is a constant and theorem is given in the following forms:

$$q_i = -k_{il}\tilde{k}(s)[p - p_f x_j g_i]_j \quad (1)$$

For q_i is given discharge vector, p is pore pressure, k is fluid absolute motion coefficient tensor, $\tilde{k}(s)$ is coefficient of the relative motion and is about liquid saturation s function, p_f is liquid density, in g_i , $i=1, 2, 3$ and g_i is the three components of the gravity vector. For saturated and unsaturated flow in FLAC^{3D}, air pressure is assumed to be constant and equal to 0.

For homogeneous isotropic solid, small deformation result from liquid flow so the liquid mass balance can be expressed as:

$$-q_{i,i} + q_v = \frac{\partial \epsilon}{\partial t} \quad (2)$$

And q_v is Body effusion source strength $\left[\frac{1}{\text{sec}} \right]$, ϵ is change of liquid volume or change of porous media unit volume liquid volume.

Liquid volume variables change are linked to pore pressure p , saturation s , mechanics volumetric strain ϵ , temperature t . Using the equation to describe the pore fluid flow and the equation is :

$$\frac{1}{M} \frac{\partial p}{\partial t} + \frac{n}{s} \frac{\partial s}{\partial t} = \frac{1}{s} \frac{\partial \epsilon}{\partial t} - \alpha \frac{\partial \epsilon}{\partial t} + \beta \frac{\partial T}{\partial t} \quad (3)$$

And M is Biot modulus $\left[\frac{N}{M^2}\right]$, n is porosity, α is Biot coefficient, β is thermal coefficient under no drainage. In the expression of FLAC^{3D}, capillary pressure effects are ignored, and the pore pressure is equal to 0 in the non-saturation region. Liquid relative migration through the cubic law and saturation build relationships, saturation is 0, full saturation is 1. Liquid flow in the unsaturated zone is only driven by gravity.

$$\hat{k}(s) = s^2(3 - 2s) \tag{4}$$

In this Formula: $\hat{k}(s)$ is permeability. S is saturation, when saturation tends to 0, penetration also tends to 0.

The boundary and initial conditions of liquid flow, the initial conditions are equivalent to a known pressure and saturation in the FLAC^{3D}. These two values must be compatible with FLAC^{3D} structure. If saturation is less than 1, the pore pressure must be 0, and the default initial pore pressure is 0, saturation is 1 in FLAC^{3D}. Considering four boundary conditions: given pore pressure, given normal component of the unit flow boundary, penetration boundary and impermeable boundary. In FLAC^{3D} default boundary is impervious, but saturation may not as a boundary condition.

The liquid mass conservation equation (2) into the pore flow constitutive equation (3) get liquid continuity equation expressions:

$$\frac{1}{M} \frac{\partial p}{\partial t} + \frac{n}{s} \frac{\partial s}{\partial t} = \frac{1}{s} (-q_{i,j} + q_v) - \alpha \frac{\partial \varepsilon}{\partial t} + \beta \frac{\partial T}{\partial t} \tag{5}$$

In the FLAC^{3D} numerical study, flow region is discretized 8-node cubic unit. Assumption: pore pressure and saturation are node variable. Internal parts are divided into tetrahedral elements and it is assumed that the pore pressure and saturation changes linearly. Therefore, in the process of numerical simulation, the groundwater level changes focus on considering the impact of saturation, pore pressure, permeability.

NUMERICAL MODEL

Beijing Subway Line 10 shuangjing station with the direction North-to-South, locates in the east of Shuangjing Bridge in East Third Ring and the cross of the East Third Ring and Guangqu Road, it is a ground three two cross (partial three-span) island platform station and the whole length 181m. Basement is on the underground one, the station hall is on the underground two, underground three is used for the platform layer.

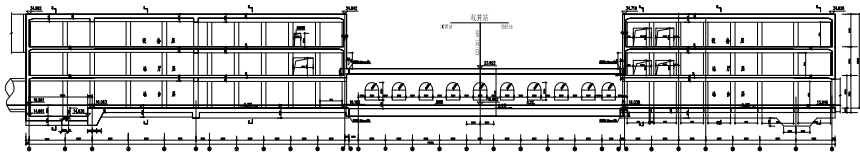
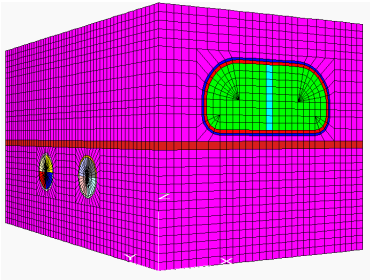


Figure 1. The Main Structure of Shuangjing Station

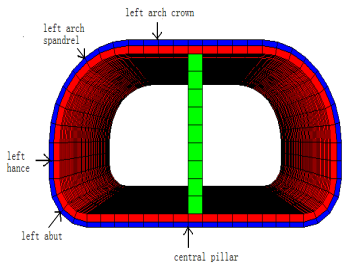
This paper mainly discusses the effect of the underwater rise to the subway station with the down through tunnels not construct and the height of station is 9.68m, the length is 53m, the initial liner is 0.4m and the second liner is 0.3. The height of middle column is 8.28m and the width is 1m, the interval space is 5m. The liner and middle column is build Elastic constitutive model and surrounding soil is build M-C constitutive model. Fix the horizontal displacement and the bottom of model, the surface is free.

Table1. Soil Parameters

soil	Width(m)	Density /Error! Reference source not found. ³	Deformation modulus /MPa	Poisson's ratio	Internal friction angle /°	Cohesion /kPa
silt	3.0	2014	14	0.25	24.8	14
Miscellaneous fill	6.2	1930	5.6	0.3	12.5	11.0
silt	4.8	2014	14	0.25	24.8	14
Silty clay	8.0	2010	5.6	0.3	12.5	11
Clay reinforced area	0.5	2200	20	0.3	40	32
sand	10.0	2000	22	0.35	32	0
Gravel	14.55	2220	90	0.3	45	0



(a). The station and the soil
Figure 2. Schematic of the Station



(b). The location of monitoring

Table 2. The Permeability Coefficient of Soil

soil	$k(m/s)$	soil	$k(m/s)$	soil	$k(m/s)$
Clay	$< 5 \times 10^{-9}$	Silty	$10^{-6} \sim 10^{-5}$	Coarse sand	$2 \times 10^{-4} \sim 5 \times 10^{-4}$
Silty clay	$5 \times 10^{-9} \sim 10^{-8}$	Sand	$10^{-5} \sim 5 \times 10^{-5}$	Gravel	$5 \times 10^{-4} \sim 10^{-3}$
Silty soil	$5 \times 10^{-8} \sim 10^{-6}$	Medium sand	$5 \times 10^{-5} \sim 2 \times 10^{-4}$	Pebble	$10^{-3} \sim 5 \times 10^{-3}$

DIFFERENT WORKING CONDITIONS

This paper mainly studies the effects when the level of underground water is 6m, 15m which is the location of arch bottom and 19.6m that the spandrel of station and surrounding soil.

DISPLACEMENT OF STATION IN DIFFERENT CONDITIONS

Because of different constitute model, the middle column has better Coordination deformation, which lead it have the biggest displacement compared to other monitoring points. From the results, it also can know that the displacement of the monitoring point is rise but not decline as normal, the reasons maybe that the soil under the station is affected by the underground water, and the pores among the soil filled with water, with the pore water pressure increase, the station will receive the action of buoyancy. The displacement when the water level is 15m and 19.6m is less than the water level is 6m for the reason that when the water level is 15m or 19.6m, the soil under the water level is not compacting, on the act of station self-weight and the soil compression performances, the station will overall sink. (Figure 3)

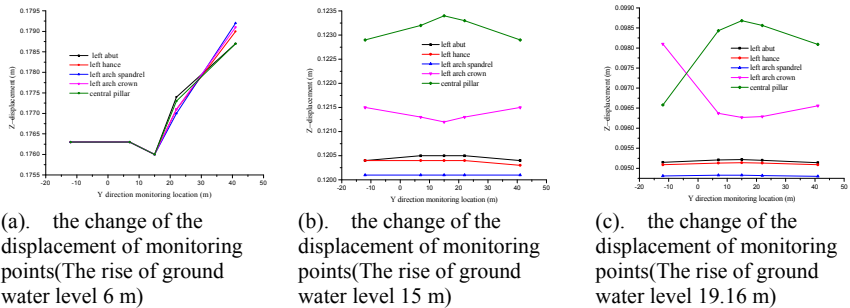
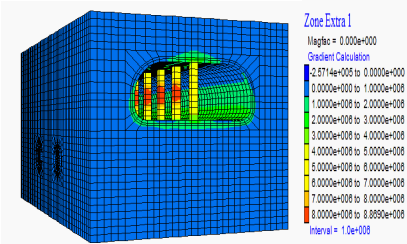


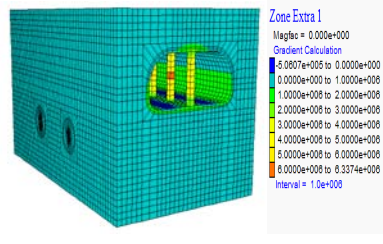
Figure 3. The Change of the Displacement of Monitoring Points under Different Water Table

ANALYSIS OF STRESS IN DIFFERENT CONDITIONS

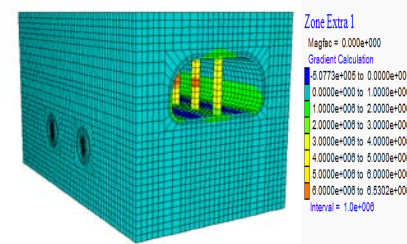
The change of principal stress in the middle column and bottom is biggest according to the results (Figure 4), when the water level is 19.16m, the difference between max principal stress and minimum principal stress is bigger than other working conditions on the bottom of middle column, the reason maybe that the station may receive more complex stress when the water level is 19.16m compare to the other working condition which make the station have an action of buoyancy(Figure 5). It also can know that the more near the middle of the station, the more obviously the stress concentration, so it is quite important to take measures to the middle column.



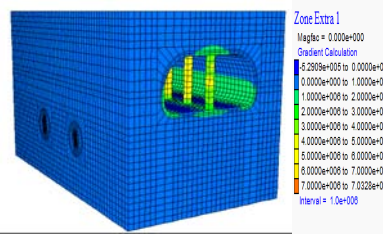
(a). The size of the principal stress difference nephogram (Line 10 excavation is completed)



(b). The size of the principal stress difference nephogram (The rise of ground water level 6 m)

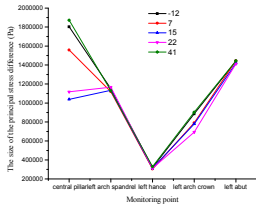


(c). The size of the principal stress difference nephogram (The rise of ground water level 15 m)

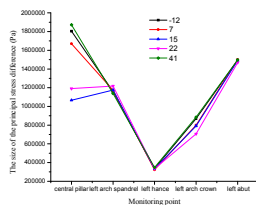


(d). The size of the principal stress difference nephogram (The rise of ground water level 19.16 m)

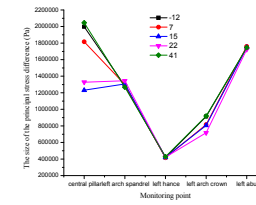
Figure 4. The Principal Stress Difference Nephogram when Groundwater Level Change



(a). The principal stress difference (the rise of ground water level 6 m)



(b). The principal stress difference (the rise of ground water level 15 m)



(c). The principal stress difference (the rise of ground water level 19.16 m)

Figure 5. The Size of the Principal Stress Difference Curves

CONCLUSION

Because the change of water level, the subway station will overall floating or sinking, and the station is unlikely damaged as the whole, the stress may concentrate in some part of the structure and make the structure crack. When the underground water raised, the middle column and the bottom of tunnel is the weakest part which should give more attention.

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Calculation Evaluation of High Speed Railway Operation Safety

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ABSTRACT

On basis of the accident data of some railway administration (from October 2011 to January 2013) and another railway administration (from January 2011 to December 2012), summarized the main influence factors of both ordinary and high-speed railway operation safety. Then analyzed characteristics of high speed railway operation safety furthermore, High speed railway operation safety has a high demand of the management level, and has more traffic safety accidents owing to the management but few accidents owing to the outside factor out of the high-speed railway system, many accidents involve in vehicle system, communication signal system and emergency rescue system. Then it constructed high-speed railway operation safety evaluation model by uncertain multi-attribute method. It took railway administration bureau A as example and concluded its operation safety situation is general in the end.

INTRODUCTION

The Chinese high-speed railroad has made great progress during the past several years, till now, some high-speed lines had operated for several years, in fact they have also had the significant security accidents in recent years of operation process, for instance the accident on July 23, 2011, how avoid such serious accident again, it is necessary to carry on the appraisal and the analysis to the past operation security, this article embarks from the massive security operation data and carries on the computation analysis the high-speed railroad security condition for future safety.

1 HIGH-SPEED RAILROAD ACCIDENT FACTOR ANALYSIS.

This article takes the Railroad Bureau A from January, 2011 to October, 2012 and the Railroad Bureau B from January, 2011 to December, 2012, each kind of railway accident and question as a background, carries on the reorganization and the analysis to the accident factor.

In the near two years, in the Railroad Bureau A ordinary railway system, 689 accidents altogether took place. According to the accident category, the security accident of factors outside the railroad is 52% of the total, accounts for the biggest proportion, the second biggest accounts for 28% for the traffic safety, the third accounts

for 20% for the labor safety. on another hand, high-speed accident in railroad bureau A add up to 62, divided according to the accident category, the traffic safety accident accounts for 89%, it is the biggest proportion, accident owing to the outside system accounts for 10%, accident related to workers accounts for 1% .

For Railroad Bureau A, if accidents are divided according to the accident factors, it is like Figure 1 and Figure 2 shows.

By Figure 1, in Railroad Bureau A, ordinary railway accident factors involve the management level which accounts for the biggest proportion, i.e. 26%; Next is the staff thought condition, some 554, accounts for total accidents 25%; the third biggest factor is the staff working ability, accident factors also involve the passenger, the pedestrian and the social vehicles' safe consciousness, the staff training, the vehicles system, the natural environment, the track, the correspondence signaling system, the power supply system and the emergency recovery accident, the social environment, the construction, the staff living standard, the staff physiology condition and psychology condition and so on.

By Figure 2, in Railroad Bureau A, high-speed railway accident factors involve the working ability, there are 27accidents caused by the working ability, which accounts for the biggest proportion, i.e. 18%; next is the management level, some 24 accidents, accounts for total accidents 16%; the third biggest factor is the thought condition and the vehicles system, both 22 accidents, respectively accounts for 15%. Accident factors also involve the emergency recovery, the safe consciousness, the training, the power supply system, the correspondence signaling system, the track, the natural environment, the construction, the social environment, the staff psychology condition and the living standard etc.

For Railroad Bureau B, ordinary railroad system altogether has 447accidents, divided according to the accident category, accident caused by outside the road occupies 64%, accounts for the biggest proportion, next train operation factor accounts for 31%, the third the labor safety factor accounts for 5%. In Railroad Bureau B high-speed railroad altogether has 31 accidents, completely involve the train operation factor.

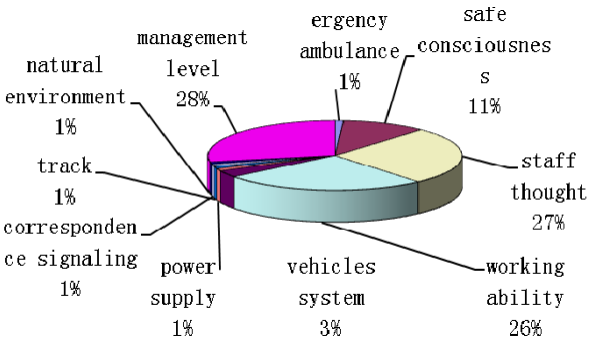


Figure 1. For Railroad Bureau A Factor Proportion of Ordinary Railway Accident

There are many factors influencing safety, if accidents divided according to the factor, like Figure 3 and Figure 4 shows.

By Figure 3, in Railroad Bureau B ordinary railway accidents, accidents quantity involved the management level is 406 pieces, accounts for the biggest proportion, 28%; Next is the staff thought condition, some 369 pieces , 25%; the third biggest proportion is 19%, which involves the passenger, the pedestrian and the social vehicles safe consciousness, some 295 accidents; Factors also involve the staff working ability, education training, the vehicles system, the track, the power supply system, the staff physiology condition, the staff psychology condition, the staff living standard, the correspondence signaling system, the construction, the natural environment, the social environment and the emergency recovery system, and so on.

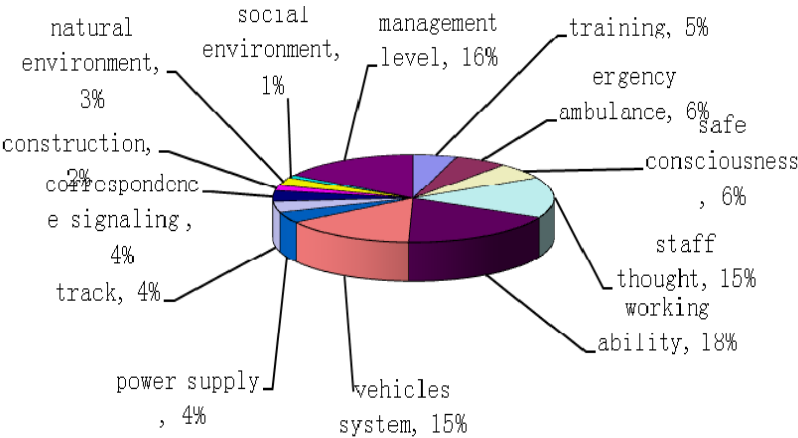


Figure 2. For Railroad Bureau A Factor Proportion of High-Speed Railroad Accident

By Figure 4, in Railroad Bureau B high-speed railroad accidents, accidents quantity involved the management level is 20, accounts for the biggest proportion, 21%; Next is the working ability, some 16 pieces, 17%; the third biggest proportion is 10%, which involves the thought condition and the correspondence signaling system, respectively 10 accidents. Factors also involve education training, the vehicles system, the power supply system, the construction, the emergency recovery system, the track, the natural environment, the social environment, safe consciousness, the staff physiology condition, the staff psychology condition, the staff living standard, and so on.

Unifies above analysis, the factors which influence high-speed railroad operation security include the passenger pedestrian and social vehicles' safe consciousness, staff's physiological condition, staff's psychological condition, staff's thought condition, staff's working ability, staff's living standard, the vehicles system, the power supply system, the correspondence signaling system, the track, the construction, the natural environment, the social environment, the management level, the training level, emergency recovery ability and so on.

2 HIGH-SPEED RAILROAD SECURITY OPERATION CHARACTERISTIC

Through the comparison high-speed railroad accidents with the total railway accidents in the last few years, the characteristic of high-speed railroad operation security are summarized as follows:

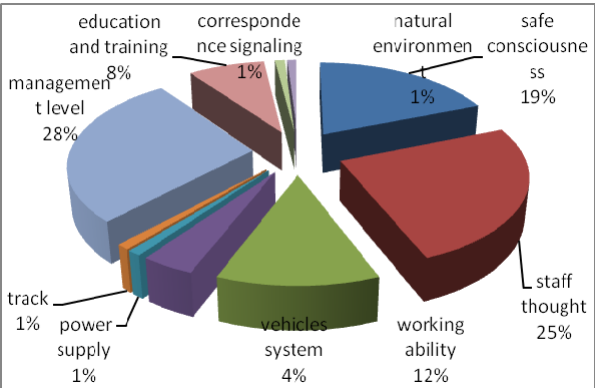


Figure 3. Railroad Bureau B Ordinary Railway Accident Factor Proportion

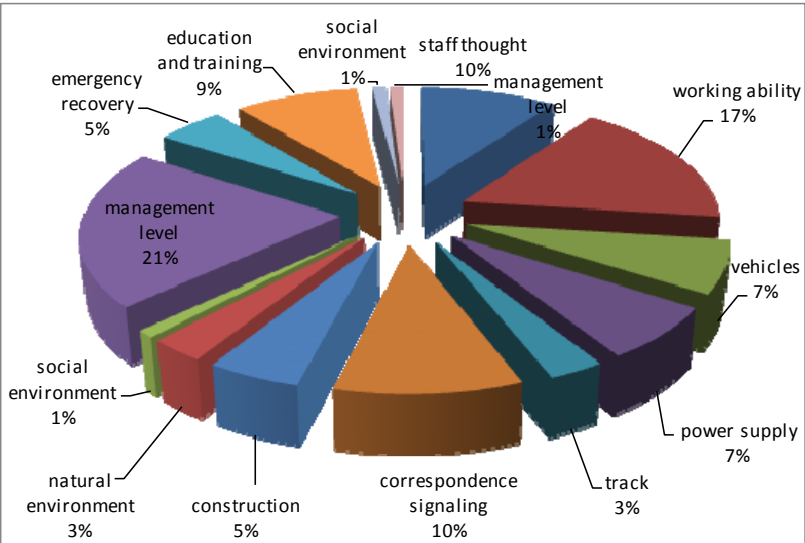


Figure 4. Railroad Bureau B High-Speed Railroad Accident Factor Proportion

(1) High-speed railroad operation security influence surface is broad. Once some place has had the train operation accident, it is easy to affect a whole line even area operation, if more serious, it can affect the entire operation system.

(2)The operation safety involves more persons and more working procedures.

In the high-speed railroad operation process, each person must obey regulations strictly, and must maintain disciplines during each task, any behavior violates regulation or a discipline has the possibility to initiate the train operation accident or the person casualty.

(3) Influences to the operation security by the natural environment and the social environment are not allowed to neglect. For example, in the flood season, the landslide, fall the stone, the mud-rock flow and so on may create the mountain mass landslide, may destroy the line, the bridge, and threaten operation security. In addition, the high-speed railroad has a lot of stations with long lines; the social macro-environment also can have the quite tremendous influence to the train operation.

(4) The high-speed railroad operation is deeply influenced by time factor. The high-speed railroad train must depart on the time which stipulated according to the train schedule for safe, and must run on schedule strictly. Sometimes even a minute second difference also possibly causes serious accident's occurrence.

(5) The high-speed railroad operation safely requests the high management level. Through analysis on the high-speed railway accidents, it is not difficult to discover that, the passive high-speed railroad operation organization control and the management fault are the key aspects of high-speed railroad operation accidents. It is necessary to take the coordination and the control measures on these management factors, to avoid high-speed railroad operation accident occurrence.

(6) There are more train operation accidents in high-speed railway than ordinary railway. From the high-speed railroad accident data analysis, in Railroad Bureau A, train operation accident occupies 28% in ordinary railway system, while in the same Railroad Bureau high-speed railroad train operation accident occupies 89%. For Railroad Bureau B, train operation accident occupies 31% in ordinary railway system, while in the same Railroad Bureau high-speed railroad train operation accident occupies 100%.

(7) There are fewer accidents caused by outside the high-speed railroad factors or workers faults in high-speed railway than ordinary railway. From the data analysis, in Railroad Bureau A, accident caused by outside the high-speed railroad factors or workers faults occupies 52% or 20% in ordinary railway system, while in the same Railroad Bureau high-speed railroad 10% or 1%. For Railroad Bureau B, accident caused by outside the high-speed railroad factors or workers faults occupies 64% or 5% in ordinary railway system; while in the same Railroad Bureau high-speed railroad both are 0%.

(8) There are more accidents involves the vehicles system, the correspondence signaling system and the emergency recovery system in high-speed railway than ordinary railway. From the accident data, in Railroad Bureau A, accident involves the vehicles system, the correspondence signaling system and the emergency recovery system occupies 5% in ordinary railway system, while in the same Railroad Bureau high-speed railroad train operation accident occupies 25%. For Railroad Bureau B, accident involves the vehicles system, the correspondence signaling system and the emergency recovery system occupies 4% in ordinary railway system, while in the same Railroad Bureau high-speed railroad 22%.

3 HIGH-SPEED RAILROAD OPERATION SAFETY EVALUATION MODEL

We divide the high-speed railroad operation safe influence factors into five parts, they are passenger pedestrian and social vehicles safe consciousness, staff condition (including staff physiology condition, staff psychology condition, staff thought condition, staff working ability and staff living standard), equipment facility (including vehicles system, power supply system, correspondence signaling system, track system and construction), environment (including natural environment and social environment) and management (including management level, education training and emergency recovery).

By above analysis on Railroad Bureau A and Railroad Bureau B accident factors, three factors of the staff physiology condition, staff psychology condition and the staff living standard have little influence on safety, here, when calculating evaluation, we neglect them; in high-speed railway system, there is hardly plain crossing situation, so social vehicles seldom do effect, transfer the passenger and the pedestrian and social vehicles safe consciousness to passenger and the pedestrian; then we collect equipment sub-factors for the equipment facility factor. Finally obtained high-speed railroad operation security appraisal target system is as shown in Figure 5.

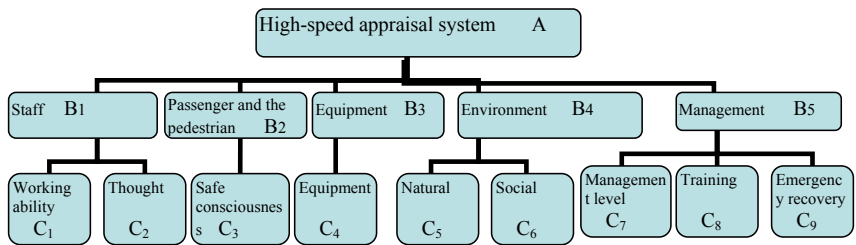


Figure 5. High-Speed Railroad Operation Safety Evaluation System

The more accident proportion, we give the more factor target weight. Moreover, when experts carries on the comparison to some targets to give judgment matrix, possibly appears cannot make the situation which the accurate judgment or the judgment is not in accordance with the facts, therefore in expert's judgment matrix possibly can appear the reciprocal judgment matrix, the fuzzy supplementary judgment matrix, the incomplete judgment matrix or the mix judgment matrix.

This article uses the characteristic vector method and the most small squares method of difference separately carries on the reciprocal judgment matrix and the fuzzy supplementary judgment matrix weight calculates. For the incomplete supplementary judgment matrix, the mix judgment matrix and the accident factor weight computation, uses reference literature 1 method to carry on the weight computation.

Consisting of accident factor weight computation: Through counts the high-speed railroad operation security accidents in section of times, analyzes influence factor and accident quantity which it occurs. If accident quantity involves the target B_i

is b_i ($i = 1, 2, \dots, n$), Carries on normalized processing to the vector. $b = (b_1, b_2, \dots, b_n)^T$

$$\omega_i = \frac{b_i}{\sum_{j=1}^n b_j}, (i = 1, 2, \dots, n) \tag{1}$$

Then $\omega = (\omega_1, \omega_2, \dots, \omega_n)^T$ is the quantitative each appraisal target weight.

If there are t-1 persons as experts to carry on the target weight judgment matrix, supposes determined this weight $\omega = (\omega_1, \omega_2, \dots, \omega_n)^T$ was the expert with number t, so there are t experts.

There are different weights for different experts. Here, we use the weighted average algorithm to calculate the appraisal targets' comprehensive weight.

Finally, computation of synthetic evaluation value divided into two steps: First step with the weighting arithmetic mean method synthesis identical expert's each weight target; Second step synthesizes each expert's appraisal value with the combination weighting arithmetic mean method, like this can transform the multi-attribute target appraisal to a quality synthetic evaluation value appraisal.

4 EXAMPLE

Based on the high-speed railroad operation safety evaluation target system (i.e. Figure 3-1), invites four railway transportation experts for Railroad Bureau A to carry on the relative importance of high-speed railroad operation safe various targets.

Supposes the weight vector which the accident factor determined is by the fifth expert gives. Then the accident factor the weight vector is $B5 = (b_1, b_2, \dots, b_9) = (20, 28, 9, 43, 5, 2, 23, 8, 9)$.

Using characteristic vector method computes weight vector for supplementary judgment matrix B1, gets:

$$\omega^{(1)} = (0.189\ 1, 0.085\ 2, 0.018\ 2, 0.119\ 3, 0.031\ 1, 0.023\ 6, 0.292\ 4, 0.182\ 7, 0.058\ 4) .$$

Calculates the weight vector for the fuzzy supplementary judgment matrix correspondence with the most small squares method of difference, gets:

$$\omega^{(2)} = (0.255\ 6, 0.155\ 6, 0.011\ 1, 0.133\ 3, 0.111\ 1, 0.033\ 3, 0.188\ 9, 0.033\ 3, 0.077\ 8) .$$

Uses the assistance matrix technique to obtain the weight vector from incomplete supplementary judgment matrix B3, gets:

$$\omega^{(3)} = (0.147\ 7, 0.121\ 0, 0.078\ 9, 0.133\ 0, 0.084\ 0, 0.069\ 1, 0.143\ 3, 0.123\ 5, 0.101\ 2) .$$

Obtains the weight vector using the linear target programming method from mix judgment matrix B4, gets:

$$\omega^{(4)} = (0.242\ 6, 0.058\ 2, 0.151\ 6, 0.145\ 5, 0.083\ 2, 0.033\ 3, 0.121\ 3, 0.106\ 1, 0.058\ 2) .$$

Considers the actual accident factor occupy portion to get the weight vector, as follows:

$$\omega^{(5)} = (0.1361, 0.190\ 5, 0.061\ 2, 0.292\ 5, 0.034\ 0, 0.013\ 6, 0.156\ 5, 0.054\ 4, 0.061\ 2) .$$

We combine expert's authoritative degree and expert's post relevance, definite expert's weight vector $\lambda = (0.25, 0.22, 0.18, 0.20, 0.15)$.

To appraise the target using weighting arithmetic appraisal operator to obtain

the appraisal target comprehensive weight sequence:

$$z(\omega) = (0.199\ 0,0.117\ 5,0.075\ 7,0.156\ 1,0.116\ 1,0.034\ 4,0.193\ 3,0.104\ 6,0.070\ 8) .$$

According to the appraisal target comprehensive weight sequence, important degree sorting of each influence factor in turn is the staff thought condition, the management level, the equipment facility situation, education training, staff working ability, the natural environment, emergency recovery ability, the passenger and the pedestrian safe consciousness and the social environment, as shown in Figure 6.

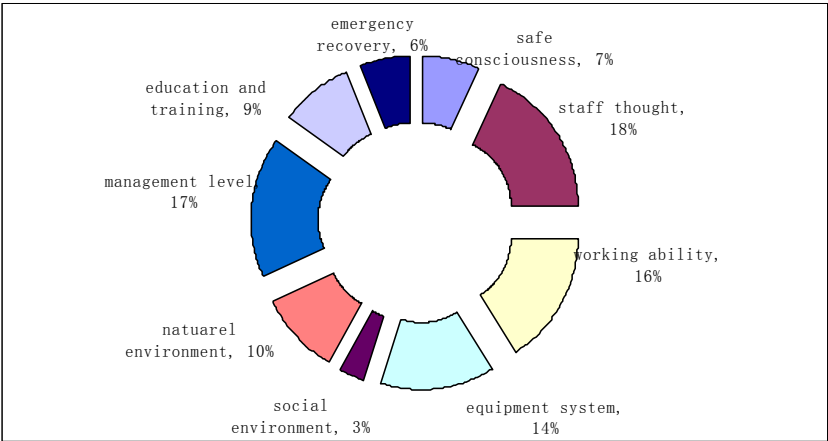


Figure 6. Appraisal Target Comprehensive Weight

After several syntheses, finally, we get goal quality synthetic evaluation value $z(\lambda, \tau) = 73.6195$.

By the quality synthetic evaluation value, Railroad Bureau A operation safe quality synthetic evaluation rank is for general. Railroad Bureau A should enhance the passenger and the pedestrian safe consciousness and in the high-speed railroad emergency recovery ability also has the insufficiency. In order to safeguard the high-speed railroad operation security, we may take more measures and set up early warning management system enhance the high-speed railroad the operation security condition.

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Transmit– Real Time Travel Time Information System in New York Metropolitan Area

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ABSTRACT

The TRANSMIT system is a regional traffic monitoring system that uses RFID tag readers to read E-ZPass tags at strategic locations throughout New York, New Jersey and Connecticut. The readers installed along or over a roadway section are used to identify vehicles equipped with E-ZPass tags provided by multi-agencies through E-ZPass Regional Customer Service Center. These vehicle tags are detected by an upstream reader and re-detected by a downstream reader, the sequential identifications of a vehicle tag allow the measurement of the travel time between the two reader locations. Through multi-agencies' initial efforts, TransCom (operators of TRANSMIT) started the Transmit system installation in Metropolitan area in 1998 with 58 pilot Transmit Readers locations on major State highway corridors. Two out of nine MTAB&T facilities (Verrazano Narrows Bridge and Robert F. Kennedy Bridge) were part of this pilot system with total of three reader sites installed at each bridge. The Transmit system provided regional the valuable real time travel time information and was expanded through each agency thereafter. The current system has more than 250 readers throughout the Metropolitan area and is still expanding.

1.0 DESCRIPTION OF THE PROJECT

MTAB&T started the Transmit design and installation at the remaining seven facilities in 2008. With the additional seven readers installation and the software enhancement to utilize the EZ-Pass Toll system data at all seven facilities, this project expands the TRANSMIT network at MTAB&T crossings, providing more data collection points to TRANSCOM (operators of TRANSMIT). It also provides the MTAB&T with valuable live travel time data from its facilities, make travel time information available to our customers via Variable Message Sign (VMS), Internet web site (<http://traveltime.mta.info>) and mobile device apps called "MTA Drive Time" as well as conduct internal traffic pattern analysis based on the historic data stored from the system.

This project substantially increased the density of the TransCom Transmit System with more accurate and detail travel time information on our facilities. The Transmit system is able to provide the traffic condition with reversible tunnel traffic operation during the peak hour; and nightly tunnel closure status via MTA Travel Time Web Site and Mobil devices Apps.

2.0 SYSTEM DEVELOPMENT

The Transmit Server is a redundant computing platform that is responsible for the following tasks:

1. Gather real-time toll lane data (including plaza E-ZPass tag information) from each plaza's computer and store the information in a database for use in travel time calculations, traffic summaries, and other data analysis/mining purposes.
2. Gather real-time E-ZPass tag information from each far-side Transmit Reader and store the information in the database for use in travel time calculations, traffic summaries, etc.
3. Fetch selected data from the database, and use it to create formatted real time traffic information messages to Advanced Traffic Management System (ATMS) Servers situated at each facility and MTA Travel Time Web Site;
4. Compute the average real time bridge crossing travel time based on the interval between the time an individual ETC (Electronic Toll Collection) tag was read by a Toll Plaza ETC reader and time it was read by the far-side Transmit Reader;
5. Provide a real-time data interface between each far-side Transmit Reader, Toll Plaza computer and TRANSCOM/TRANSMIT Servers. This data interface is to be implemented such that it emulates the TRANSCOM/TRANSMIT communications protocol.

2.1 System Hardware

The MTAB&T Transmit System servers originally consisted of *dual* servers in a redundant configuration and are currently upgraded into dual virtual machines with enhanced capacities.

2.2 Database

The MTAB&T Transmit system uses relational database for managing the data elements that it processes. The Transmit database is hosted on a server. MTAB&T Technology Department (TD) staff is responsible for database administration tasks, security, backups, etc.

2.3 Network Interface

The Transmit System readers installed on MTAB&T facilities as well as the Toll System data feed from MTAB&T Toll Plaza computer transmit the E-ZPass tag data to Transmit Servers. The network terminal servers are used to function as remote ports for Transmit Servers.

The Transmit Virtual Servers were deployed at two separate physical locations for disaster recovery purpose. They both communicate with assigned readers and plazas primarily, and will be capable of activating as a fully functioning server if one

server is not available due to any disaster condition on site.

2.4 Toll Plaza Data Splitters

MTAB&T Toll System Computers transmit lane activity data from all of its associated toll lanes. Each port only transmits lane activity data from a group of toll lanes that have been pre-defined in the system. Every Toll System Computer has its own external serial port subsystem. Originally, each of the ports carrying the lane data is connected directly to a corresponding port attached to the plaza's other existing applications.

In order to accommodate the existing applications and the new Transmit application, the "transmit" pins on each of the computer connectors must be split and fed into the "receive pins" of the connectors belonging to the other application *and* the Transmit servers. The existing data feed to other applications remains intact and operational.

In order to split the signals in a "clean", reliable, and easily maintainable manner, the 66 block (a type of punch-down block to connect sets of wires in a telephone system) was used to split the data into two systems from Toll System computer port connectors.

This effort approved to be very cost efficient to utilize the data from existing toll system to pair up the data from newly installed Transmit readers on the far side of the facilities.

2.5 Application Software

A custom software package was developed for project by E-Transit. This software incorporated the existing software modules for interfacing with the Plaza Computers, developed XML file handling, and interfacing with Transmit Readers.

The Transmit Application Server (TAS) communicates with the Transmit Readers using the "lane controller" communications protocol. This protocol passes E-ZPass tag information from the antenna(s) out to reader communication port. As a result, the Network Terminal Server is deployed with each farside reader installed on the site.

An application running in the TAS communicates directly with each of the farside Transmit Reader via the Network Terminal Server. E-ZPass tag information is collected in real-time, immediately transferred to the TRANSCOM/TRANSMIT Server, and then stored in the TAS database. The stored tag information will be used for various purposes, including tag matching for travel time calculations, XML based messaging to MTA Travel Time Web Site FTP Servers, traffic summary information, etc.

One unique feature of this project was the development and implementation of a brand new concept that we call a "Virtual" Transmit Reader. The new Plaza Virtual Reader Application software application developed in this project virtually replicated a Reader at each Plaza. This software application extracts real-time ETC tag data from each of the plaza's toll lanes across multiple ETC readers, collects "live" E-ZPass tag ID information from the Toll Transaction data generated by Plaza computer

system at each of the MTAB&T facilities; this E-ZPass tag data is re-formatted and forward to the TransCom system in real-time, as if they come directly from the “actual” Transmit Readers. The real-time information from the millions of ETC tags that pass through our toll facilities is incorporated into the TRANSCOM Regional Architecture.

3.0 TRANSMIT TRAVEL TIME SYSTEM REPORTINGS

After the MTAB&T Transmit system installation, several travel time reporting systems were developed. The MTAB&T Travel Time web site was deployed in 2010 with all ten facilities live travel time and animation traffic condition on the following web site: <http://traveltime.mta.info>.

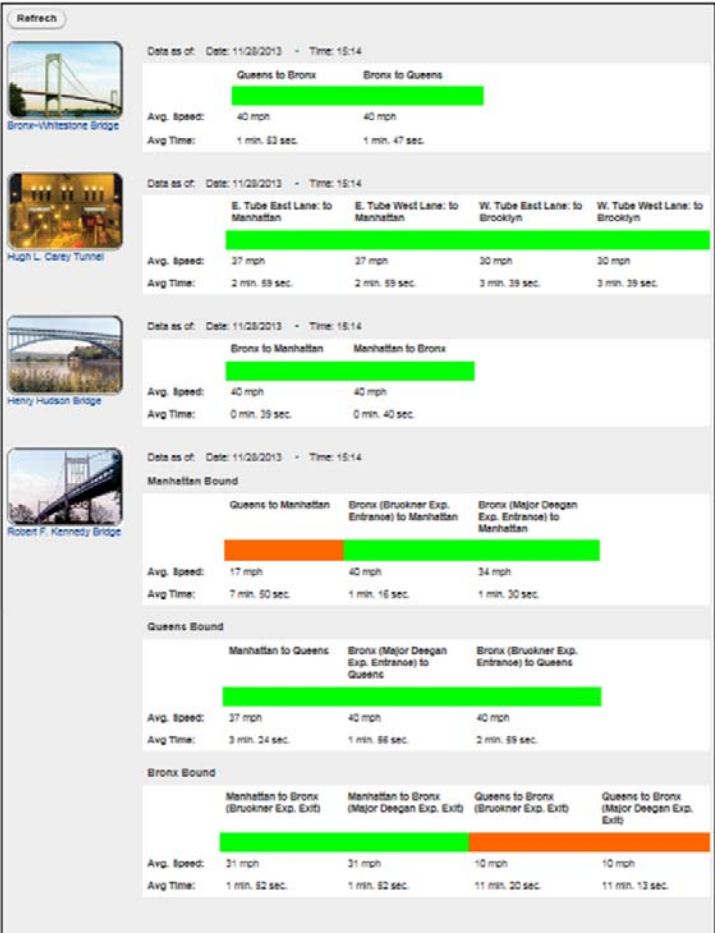


Figure1. MTA B&T Travel Time Web Site

We also developed the smart phone version of the applications (apps) for iPhone and Android phone users to download. The apps are available with name of “MTA Drive Time” on smart phone Market/Play Store for free downloading.

The smart phone apps provide the real time travel time on MTAB&T facility crossings and the travel time from each facility to other major destinations as follows:

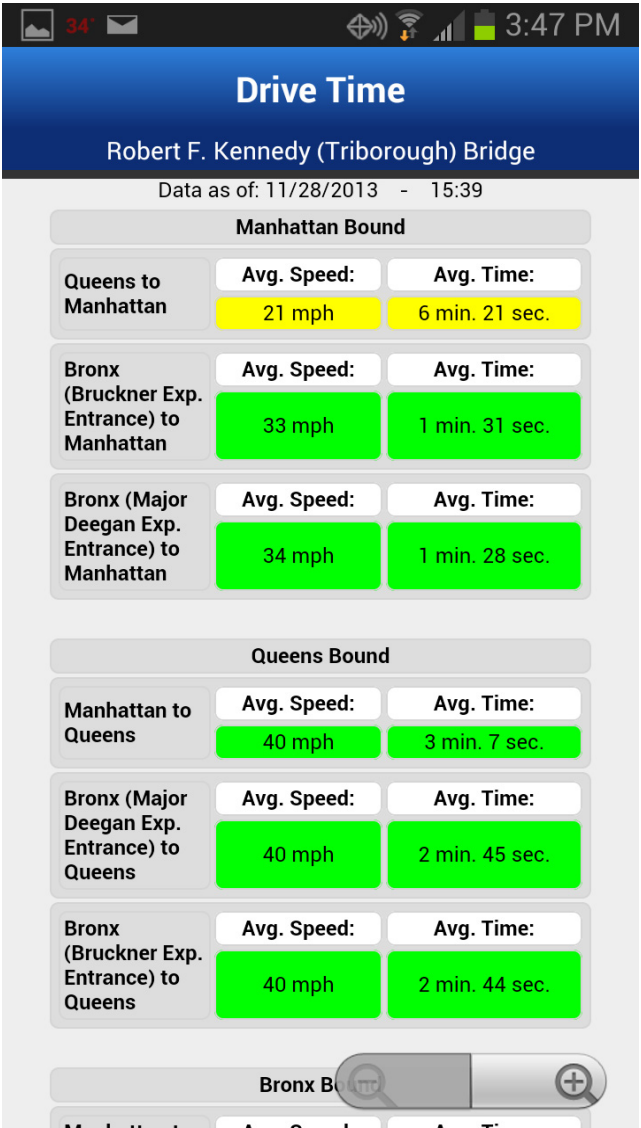


Figure 2. MTA Drive Time Apps for Smart Phone

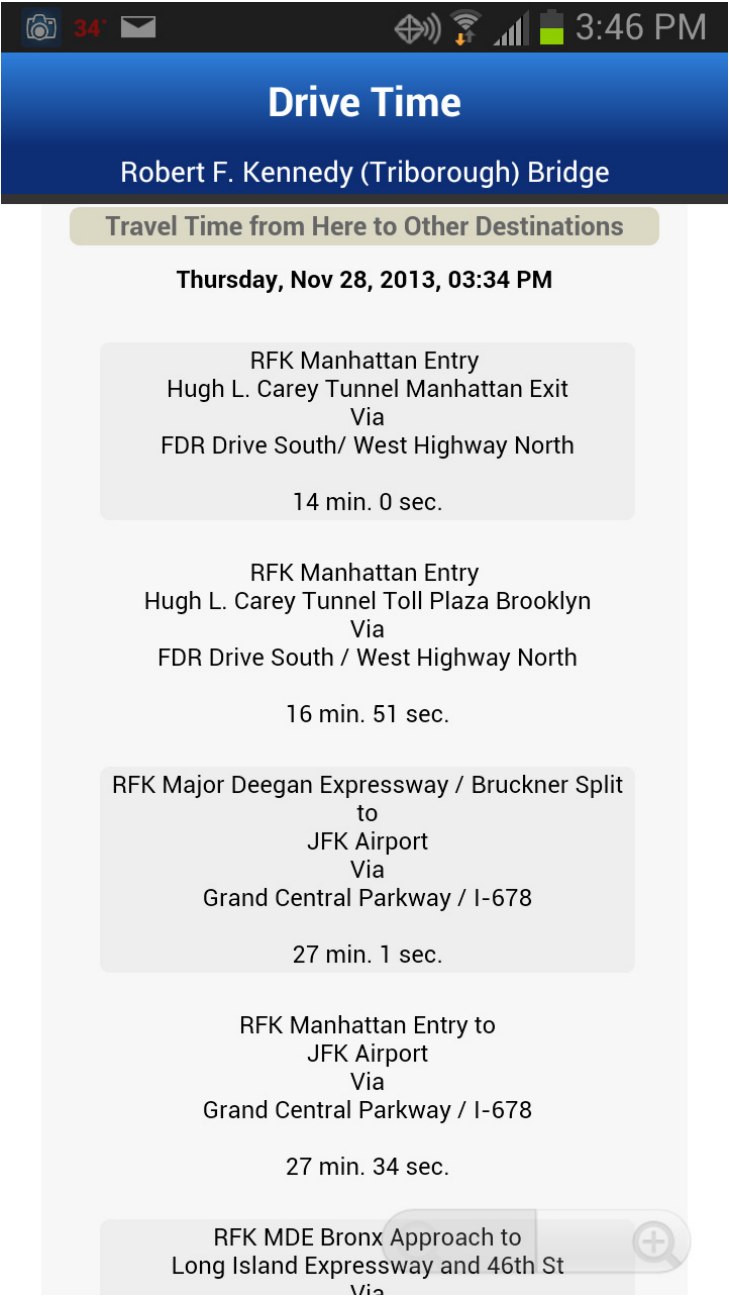


Figure 3. MTA B&T Drive Time Apps for Major Destinations from each facility

Before Transmit System, there is no toll facility travel time performance measurement available, we usually sent out the officers to drive through each facility to measure the travel time and delay on each toll plaza approach. With the Transmit System installation, the new performance reports (daily rush hour and monthly performance measurement report) are developed in house for operation to create the daily and monthly travel time reports to MTA Headquarter. The reports are indicated below.

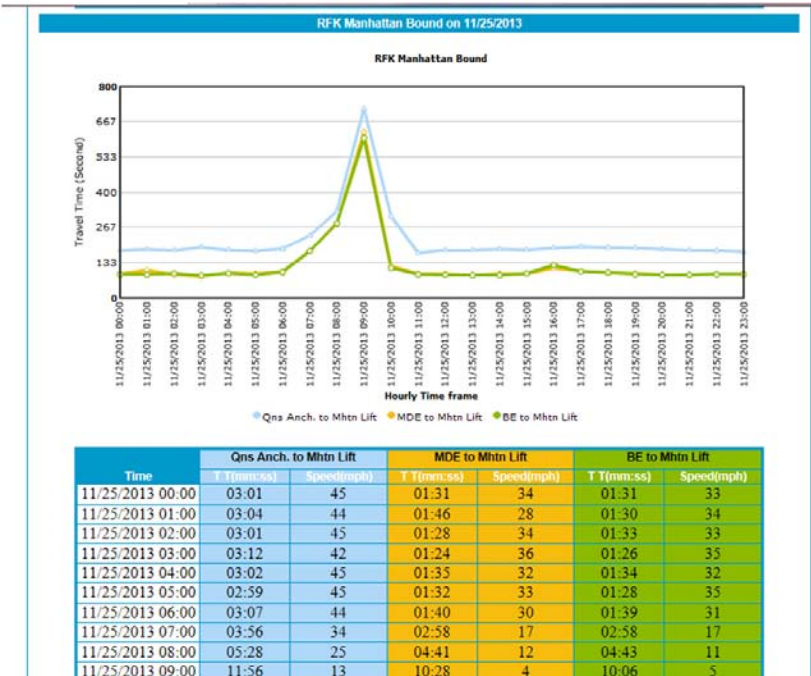


Figure 4. MTA B&T Facility Historic Hourly Travel Time Report

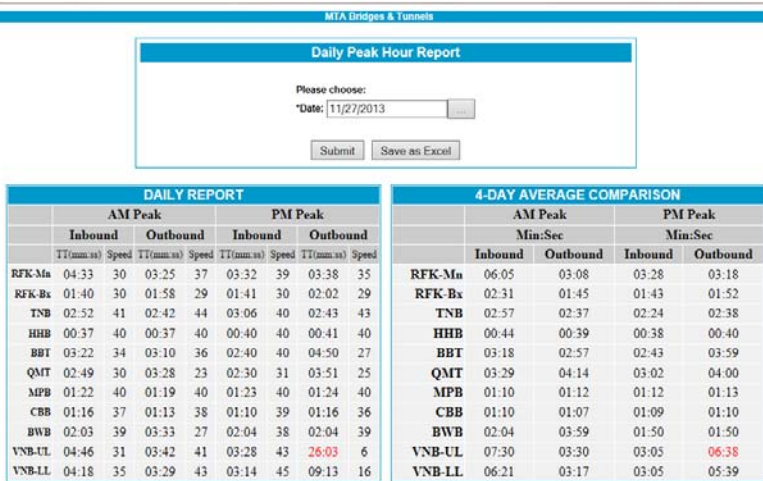


Figure 5. Daily Facility Rush Hour Average Travel Time

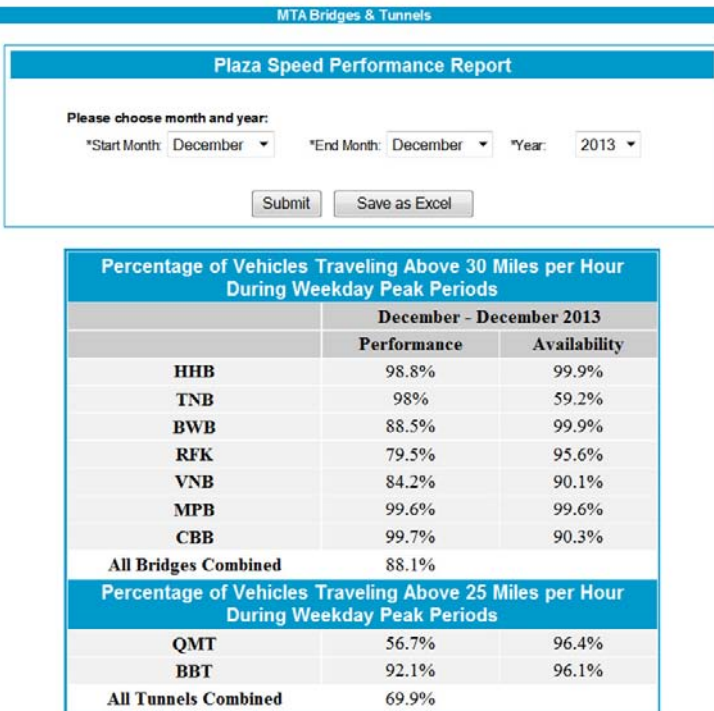


Figure 6. Monthly Performance Report

4.0 SUMMARY

The MTAB&T Transmit System provides a useful tool for our facility operation to better serve the public via real time travel time information; this system significantly enhances the information provided by the TRANSCOM Regional Architecture to public. It also substantially increased the density of Transmit locations at our plazas without additional hardware installation/cost to Authority, or modifications to the existing TRANSCOM software applications. This project contributes the better public services by providing real time travel time information to our local operation, regional transportation system.

This project was awarded Outstanding ITS Project of the Year by Intelligent Transportation Society of New York in 2010 and Smart Solution Spotlight Award from ITS America in 2011.

In addition, we are in design stage to add 11 new Transmit Readers near our facility approaches from State Highway and activate 3 toll plazas data feed from Robert F. Kennedy bridge and Verrazano Narrows bridge. This expansion will provide our operation with travel time and delay information from State highways to our ten Toll Plaza approaches; allow operation better manage the Toll/Cash lanes with the real time traffic and travel time conditions on the toll plaza approaches.

Integrated User-Based Traffic/Travel Information through Open-Architecture, Public-Domain, Cloud Computing

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ABSTRACT

This paper describes how Taipei City's ITS/ATIS development experience provides an excellent ITS example on how user-oriented, advanced ITS services can be designed, developed, implemented, and deployed with available information communication technology (ICT) by keeping the end-user in mind in world-class cities. There is an immediate need in many megacities in Asia and throughout the world to elevate traffic management, system control, and operational practice to the next level by maximizing the system, equipment, and personnel already in place. Since most cities and counties are now well-equipped with new control centers, signal systems, and capable operational staff, the results can be significant. Implementation guidelines, real-time data platforms, and information exchange standards are readily available in Taipei for potential implementation elsewhere to improve and deliver sustainable transport in the 21st century.

INTRODUCTION

Significant development has been made in the past several decades by many operating agencies throughout Asia and the world in deploying various Intelligent Transportation Systems (ITS). In Taiwan, the city of Taipei has been successfully implementing sustainable urban green transport with low carbon emissions since the 1990s that are built across intelligent transport, public transport, safety enhancement, and seamless transport for all citizens, including minority groups, persons with disabilities, and older adults. In addition to various "last-mile service hardware" investments (mini-bus, taxi, walk, bike) to facilitate public transport use, recent ITS/ATIS system integration has successfully delivered transport-related information on smart phones (iPhone, Android, Windows Mobile), which have integrated and delivered real-time traffic and travel information within and outside the Taipei metropolitan area, as shown in Figure 1.

There is an immediate need in many megacities in Asia and throughout the world to elevate feasible traffic management, system control, and operational practice into the next level by maximizing the capabilities of systems, equipment, and personnel already in place. This paper describes how the city of Taipei's ITS/ATIS development experience can provide an excellent ITS example and transfer to other places on how user-oriented, advanced ITS services can be best designed, developed, implemented, and deployed for improved service. Implementation guidelines, real-time data platforms, and information exchange standards are readily available in

Taipei for implementation elsewhere to improve and deliver sustainable transport through information communication technology (ICT) by keeping the end-user in mind in world-class megacities in the 21st century.



Figure 1. Integrated Taipei City transportation network.

DEVELOPMENT STRATEGY

Significant development has been made in the last decades in deploying urban ITS and implementing various ICT for smart city/digital city implementation by many operating agencies throughout the world, including Taipei. These include the deployment of various types of vehicular/pedestrian detector systems, advanced traffic signal controllers, wire-line and wireless communication systems, high-speed information exchange networks, cloud-based computing, and integrated software/information platforms since the 1990s.

Successful third-party application development (apps) and open standards used in “Taipei Easy Transport” was demonstrated at the 2010 Taipei International Flower World Trade Festival and was greatly enhanced in follow-up activities in 2011–2013, as show in Figure 2. This successful system development and implementation proves that (1) successful ITS/ATIS service is demanded by all travelers, commuters, and tourists, (2) feasible Public-Private Partnerships (PPP) are possible through open-architecture, open-standard data exchange platforms, and (3) innovative ATIS service can be delivered through personal communication devices (iPhone, Android, Windows Mobile).



Figure 2. Taipei Easy Transport, downloadable from the Internet.

Implementation guidelines, information exchange standards, and, most of all, the implementation experience are readily available in Taipei for implementation elsewhere. However, efforts are still needed to (1) devise implementation packages, (2) develop education/training packages, (3) devise evaluation mechanisms, and (4) implement web-based Community of Practice (COP) and peer to-peer supports to improve and deliver sustainable transport. There is an immediate need in many megacities throughout the world to elevate traffic management, system control, and operational practice to the next level by maximizing the capabilities of systems, equipment, and personnel already in place. Since most cities and counties are now well-equipped with new control centers, signal systems, and capable operational staff, the results can be quite significant.

DEVELOPMENT TO DATE

Significant deployment has been made, in the last decades in deploying the foundation of urban ITS by many operating agencies throughout Asia and the world. In Taiwan, Taipei, through its advanced sustainable, green urban transport, has successfully integrated the needed real-time transport/traveler information service by integrating intelligent transport, public transport, safety enhancement, and seamless transport for all citizens, including minority groups, persons with disabilities, and older adults. Beside improving “last-mile service” investments (mini-bus, taxi, walk, bike) to facilitate green/public transport usage, the ITS/ATIS system integration has successfully delivered transport-related information on web pages, smart phones (iPhone, Android, Windows Mobile), and information kiosks within and outside the Taipei metropolitan area.

“Taipei Easy Transport” (shown in Figure 3) has been successfully developed, instrumented, and maintained by the City of Taipei Traffic Bureau since 2009 and includes real-time bus location information (4,100 buses, 101 smart bus stops, 460

electronic bus signs, and 46 kiosks), up-to-date parking information (car, motorcycle, bike), real-time share-ride bikes, real-time taxi calls, a bus/MRT/LRT/ferry network, and real-time traffic surveillance information (speed, CCTV images, accidents in the past two months) on all freeway, expressway, and street networks within Taipei metropolitan areas.



Figure 3. Taipei Hao Xing (Taipei Easy Transport).

At present, current use is extremely significant since many users can now roam Taipei City through its well-established subway/MRT/LRT/bus/bike network with virtually the same, up-to-date, real-time information as if they were sitting at the Taipei Traffic Management Center by using the well-established WiFi, 3G, 4G network within Taipei City. As shown in Figure 4, the most popular ITS/ATIS inquiries rank include mass rail transit, light rail transit, bus rapid transit, eBus, bike, traffic surveillance, and parking information. Taipei has an innovative, updated open-architecture, public domain, open-standard, cloud-based computing data sharing policy/exchange. The platform has not only successfully integrated and delivered all

travel and traffic information to its end users but has also spurred new ITS/ATIS development efforts (involving 72 agencies and third-party software developers) and has implemented various marketable and bundled apps (64+) (for cell phones, webs, TV news reporting, navigation devices, R&D projects, etc.) that are built on top of existing government system/archived data. Taipei ITS/ATIS service would not be possible without the sustainable ITS deployment and basic urban infrastructure investment since 1990s.



Figure 4. Successful real-time usage and third-party development.

CONCLUSIONS

Significant development has been made in the last decades by many operating agencies throughout the world, including Taipei, in deploying urban ITS. These include the deployment of various vehicular/pedestrian detector systems, traffic signal controllers, communication systems, information exchange networks, and standard central computer software platforms. The successful use and apps of “Taipei Easy Transport” were demonstrated at the 2010 Taipei International Flower World Trade, and follow-up activities that showed (1) successful ITS/ATIS service is demanded by all travelers, commuters, and tourists, (2) feasible public-private partnership (PPP) development can be accomplished through open-architecture and open-standard data exchange platform/service, and (3) ATIS service can be best delivered through personal communication devices that are readily available today.

There is an immediate need in megacities throughout the world to elevate the traffic management, system control, and operational practice to the next level by maximizing the capabilities of systems, equipment, and personnel already available. Since most cities and counties are now well-equipped with new control centers, traffic controllers, and operational staff, the results can be significant. Various implementation guidelines and information exchange standards are available in Taipei and Taiwan for potential, feasible implementation throughout the world. However, efforts are needed to (1) devise a practical implementation framework, (2) develop education and training packages, (3) devise practical evaluation mechanisms, and (4) develop a web-based Community of Practice (COP) and peer-to-peer support network to improve the skills for developing and delivering sustainable, green transport.

The city of Taipei's ITS/ATIS development experience provides an excellent ITS implementation example on how various ITS services can be designed, developed, implemented, and deployed with current ICT and with keeping the end-user in mind in any international, world-class city in the 21st century.

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Determining the Impact of Gender and Age on Drivers' Acceptance of Graphic-aided Portable Changeable Message Signs in Highway Work Zones

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ABSTRACT

Graphic-aided portable changeable message sign (PCMS) is an innovative temporary traffic control device that has been proven in recent studies to improve drivers' response more effectively than the traditional text-based PCMSs. This study investigated the impact of drivers' gender and age on their acceptance of using graphic-aided PCMSs in highway work zones. A total of 429 driver surveys were conducted in a real-world highway work zone traffic condition under three graphic-aided PCMSs with different settings. Chi-square tests of independence were used to determine the relationship between drivers' gender and age and their preferences to message format. The results revealed that drivers' gender and age did not have a significant impact on their preferences to message format. The results also suggested that the majority of drivers preferred to see the warning messages displayed on PCMSs in the graphic-aided format instead of the traditional text format in highway work zones. This study provided valuable knowledge to government agencies and the transportation industry on the use graphic-aided PCMSs in highway work zones.

INTRODUCTION

Portable changeable message sign (PCMS) is a type of temporary traffic control (TTC) device which is capable of displaying a variety of messages to inform motorists of unusual driving conditions (FHWA, 2003). The traditional type of PCMS is text-based and has been employed in highway work zones for decades. Many recent studies (Wang et al., 2007; Ullman et al., 2009), however, pointed out that using text messages on PCMSs has several limitations, such as confusing drivers and delaying their responses during driving, being difficult to read for elderly drivers and non-English-speaking drivers, difficult to see under adverse viewing conditions, and having a short range of legibility. As sign technology advances, the full-matrix PCMS has made it possible to display graphics to drivers. Graphic-aided messages on PCMSs could likely overcome some of these limitations, particularly in complicated

driving conditions such as work zones (Ullman et al., 2009). Although the advantages of graphic-aided messages were realized, their use on PCMSs is still new in the U.S., and few and limited research studies on graphic-aided PCMSs have been found. This study investigated the impact of drivers' gender and age on their acceptance of using graphic-aided PCMSs by conducting driver surveys in a real-world highway work zone traffic condition under different graphic-aided PCMSs.

BACKGROUND

With the help of advanced technology in the recent decade, most studies on graphic-aided PCMSs have been conducted in simulator environments. Wang et al. (2007) assessed the effects of adding graphics to PCMSs using a driving simulator, and concluded that most people preferred graphic messages to text messages, and graphic messages were responded to significantly faster than text messages, particularly for elderly drivers and non-native drivers. Ullman et al. (2009) studied drivers' understanding of the work zone graphic using laboratory instruments, and revealed that while both text and symbol representations of roadwork were well understood by over 80% of the participants, the roadwork symbol was better understood than text by the Spanish-speaking participants and should be used in graphic designs.

Simulation studies in laboratory environments, however, have some limitations. Their results only provided participants' message reading performance in optimal circumstances. In the real-world driving situation, on the other hand, drivers have to pay attention to many other tasks, such as lane keeping, speed controlling, and car following (Huang and Bai, 2012). To overcome the limitations of simulation studies, this study employed driver surveys in a real-world work zone traffic condition to determine the impact of drivers' gender and age on their acceptance of using graphic-aided PCMSs in highway work zones.

DESIGN

A full-matrix PCMS was programmed to display a text message and three graphics. The text message was WORKZONE | AHEAD | SLOWDOWN. The graphics included three work zone graphics, designed similar to the W21-1 sign specified by the Manual on Uniform Traffic Control Devices (MUTCD), as shown in Figure 1.

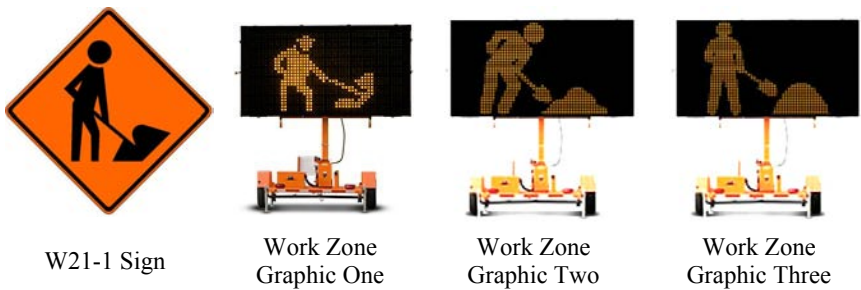


Figure 1. Work Zone Graphics in MUTCD and on PCMS

Using the text message and three graphics, three PCMS conditions were set up, which were Work Zone PCMS One (see Figure 2), Work Zone PCMS Two (see Figure 3), and Work Zone PCMS Three (see Figure 4). Each of the PCMSs had two phases, one displaying a text message and the other displaying a graphic, and it took three seconds to switch from one phase to the other. The PCMSs were placed 575 ft upstream of the beginning of the work zone, which was defined by the W20-1 sign ROAD WORK AHEAD.



Figure 2. Work Zone PCMS One



Figure 3. Work Zone PCMS Two



Figure 4. Work Zone PCMS Three

Driver surveys were conducted in a work zone located on US-75 between Burlington, KS and I-35. The construction project was paving operations to rehabilitate the roadway surface. When construction operations were underway, the two-lane highway was converted to a one-lane, two-way work zone. A flagger was used at each end of the work zone for traffic control, and a pilot car was employed to guide through traffic. All passing vehicles had to stop before the flagger and wait for the pilot car, which typically took 10 to 15 minutes. Driver surveys were conducted at the flagger locations during drivers' waiting time without interrupting traffic and causing further traffic delay. A single survey took up to three minutes in most cases.

The driver survey was designed to determine drivers' preferred format of warning messages displayed on a PCMS in highway work zones. Surveyed drivers were asked to choose their preferred format of warning messages displayed on the PCMS from *Text format*, *Graphic-aided format*, *No difference*, *Don't care*, and *Don't know*. Drivers were also asked about their age information from the following seven age groups: *Less than 19*, *19 to 24*, *25 to 34*, *35 to 44*, *45 to 54*, *55 to 64*, and *Over 65*. In addition, drivers' gender was recorded as *Male* or *Female*.

METHODOLOGY

Chi-square tests of independence were used to determine the relationship between drivers' gender and age and their preferences to message format on PCMS. If drivers' gender and age are statistically independent of their preferences to message format, then drivers in all gender and age groups would have statistically the same preferences to message format on PCMS; otherwise, drivers in different gender and age groups would have different preferences to message format on PCMS.

In the Chi-square tests of independence, the null hypothesis (H_0) and the alternative hypothesis (H_1) were defined as:

H_0 : Drivers' gender or age is statistically independent of their preferences to message format.

H_1 : Drivers' gender or age is not statistically independent of their preferences to message format.

In other words, the interpretation of the null hypothesis is that drivers in all gender or age groups have statistically the same preferences to message format on PCMS, and the alternative hypothesis is that drivers in different gender or age groups have different preferences to message format on PCMS. A 95% level of confidence was used in the Chi-square tests, and a p-value no greater than 0.05 would indicate that the null hypothesis could be confidently rejected.

RESULTS

A total of 429 valid questionnaires were collected from the driver surveys, including 150 for Work Zone PCMS One, 139 for Work Zone PCMS Two, and 140 for Work Zone PCMS Three.

Drivers' Gender and Preferences to Message Format

109 male drivers (73%) and 41 female drivers (27%) responded to the survey under Work Zone PCMS One. Figure 5 shows the details of male and female drivers' preferences to message format. Around 10% more female drivers liked the text format, and about 9% more of male drivers chose the graphic-aided format. The difference of male and female drivers who selected other options varied within a small range of 4%. The p-value of Chi-square test of 0.730 indicated that drivers' gender was statistically independent of their preferences to message format, and that male and female drivers' preferences to message format were statistically the same under Work Zone PCMS One.

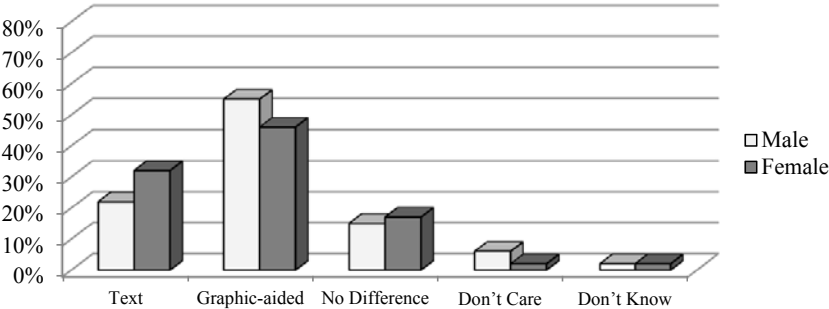


Figure 5. Drivers' Gender and Preferences under Work Zone PCMS One

78 male drivers (56%) and 61 female drivers (44%) took the survey under Work Zone PCMS Two, and their preferences are detailed in Figure 6. A similar percentage of male and female drivers chose the text format (10% to 12%) and *No difference* (14% to 15%). On the other side, about 10% more male drivers liked the graphic-aided format. The p-value of Chi-square test of 0.350 revealed that drivers' gender was statistically independent of their preferences to message format, suggesting that male and female drivers' preferences to message format were statistically the same under Work Zone PCMS Two.

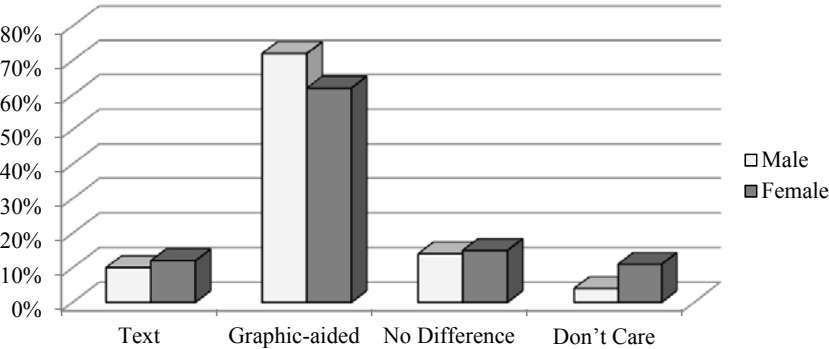


Figure 6. Drivers' Gender and Preferences under Work Zone PCMS Two

101 male drivers (72%) and 39 female drivers (28%) participated in the survey under Work Zone PCMS Three, the details of which are presented in Figure 7. The preferences of male and female drivers to the text and graphic-aided format were similar measured in percentage. The percentage of male and female drivers who chose *No difference* and *Don't care* varied within a small range as well. The p-value of Chi-square test of 0.957 suggested that drivers' gender was statistically independent of their preferences to message format, and that male and female drivers' preferences to message format were statistically the same under Work Zone PCMS Three.

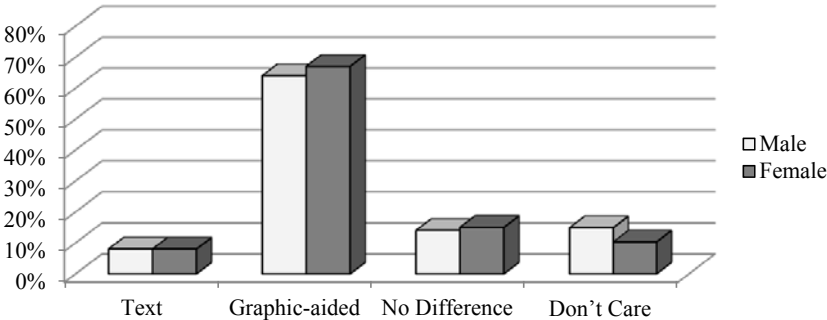


Figure 7. Drivers' Gender and Preferences under Work Zone PCMS Three

Drivers' Age and Preferences to Message Format

Under Work Zone PCMS One, surveyed drivers in 45–54 and 55–64 accounted for the two largest age groups, at around 25%, followed by drivers in 25–34 and 35–44 age groups. The percentages of young drivers below 25 and elderly drivers over 65 were less than 10% in each age group. Drivers' age and preferences to message format are detailed in Figure 8. Text format was preferred by over 20% of drivers in age groups younger than 65, but was selected only by 13% of elderly drivers over 65. Drivers in 55–64 stood as the largest age group in favor of graphic-aided format, which was over 60%, followed by 45–54 and over 65 age groups, which were over 50%. The percentage of drivers choosing the graphic-aided format in the remaining age groups varied between 44% and 50%. The p-value of Chi-square test of 0.675 suggested that the difference of percentages was not significant between age groups, and that drivers' preferences to message format were statistically independent of their age group under Work Zone PCMS One.

Under Work Zone PCMS Two, the two largest age groups were 25–34 and 35–44, which accounted for 24% and 22%, respectively, followed by 45–54, 55–64, and over 65. Young drivers below 25 were less than 10% of all respondents in each of the two age groups. The detailed drivers' age and preferences to message format are presented in Figure 9. Around 10% to 13% of drivers in all age groups preferred the text format except for drivers in 20–24. Over 75% of drivers in 15–19, 35–44, and 55–64 preferred the graphic-aided format, while the percentage of drivers in other age groups were around 60%. The percentage of drivers seeing no difference in message formats varied between 0% and 33%. The p-value of Chi-square test of 0.763 that the

difference of percentages between each age group was not significant, and that drivers' preferences to message format were statistically independent of their age group.

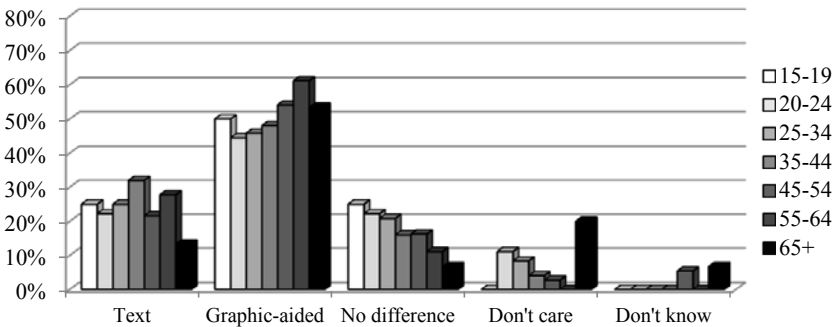


Figure 8. Drivers' Age and Preferences under Work Zone PCMS One

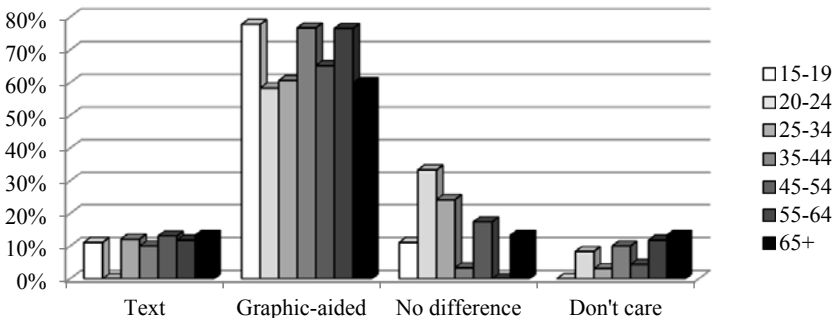


Figure 9. Drivers' Age and Preferences under Work Zone PCMS Two

Both of the age groups 45–54 and 55–64 had over 25% of drivers who responded to surveys under Work Zone PCMS Three, followed by 17% in the 25–34 age group and 12% in the 35–44 age group. The percentages of young (15–19, 20–24) and elderly drivers (over 65) were less than 10% in their age group. Figure 10 details drivers' age and preferences to message format. Except for drivers younger than 19 and over 65, around 8% to 12% of drivers in each age group chose text format. Drivers in 15–19, 35–44, 45–54, and 55–64 in favor of graphic-aided format stood as the largest four age groups, the percentages of which were over 65%. The percentages of drivers in other age groups were less than 55%. The percentage of drivers who selected *No different* and *Don't care* varied a lot in different age groups. The p-value of Chi-square test of 0.808 indicated that the difference of percentages in drivers' preferences was not significant between age groups, suggesting that drivers' preferences to message format were statistically independent of their age group under Work Zone PCMS Three.

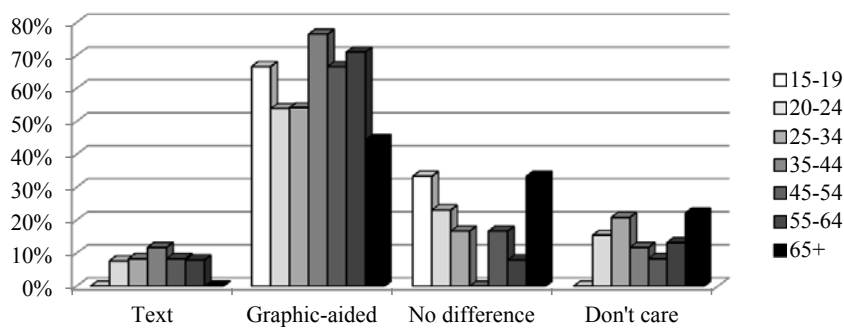


Figure 10. Drivers' Age and Preferences under Work Zone PCMS Three

DISCUSSION AND CONCLUSIONS

While most recent research on graphic-aided PCMSs were carried out in simulator environments, this study employed driver surveys in a real-world work zone traffic condition to determine the impact of drivers' gender and age on their acceptance of using graphic-aided PCMSs in highway work zones. A text message and three work zone graphics were displayed on a PCMS which was placed 575 ft upstream of the beginning of a work zone, and a total of 429 valid questionnaires were collected from the driver surveys. Chi-square tests of independence were used to determine the relationship between drivers' gender and age and their preferences to message format on PCMS. The results of driver surveys revealed that drivers' gender and age did not have a significant impact on their preferences to message format under different graphic-aided PCMS settings. The results also suggested that the majority of drivers preferred to see the warning messages displayed on PCMSs in the graphic-aided format instead of the traditional text format in highway work zones. The results of this study provided valuable knowledge to government agencies and the transportation industry on how to regulate and use graphics on PCMSs in highway work zones.

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A Multi-Objective Traffic Signal Control Model for Intersection Based on B-P Neural Networks

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ABSTRACT:

Real-time traffic counts between entry and exit of intersections are important input data for traffic signal control system, and also difficult to obtain. In on-line systems, it is impossible to detect such data directly through traffic surveillance system, and also unrealistic to gather them from conducting surveys. This paper first reviewed the evolution process of traffic signal control system, and analyzed characteristics of real-time traffic counts at intersections. Then a back-propagation neural network (BPN) model was proposed to estimate and forecast the dynamic turning movements, and an algorithm was designed to solve the model. Furthermore, based on the estimated turning movements, a multi-objective model was put forward to achieve the real-time signal control parameters and the evaluation indices. Simulation experiments and case study were implemented, and the results were compared with classical Webster method. The reported examples have clearly indicated that the models and algorithms are rather efficient and accurate, and can realize the real-time signal control for intersections.

Key words: traffic signal control; neural network; turning proportion

INTRODUCTION

With the development of intelligent transportation system, recent interests in the application of advanced technologies for communication and control have renewed in the development of real-time traffic-adaptive signal control systems.

The accurate intersection traffic turning proportion is the basis of timing a scientific scheme for the intersection. Real-time traffic counts between entry and exit of intersections are important input data for traffic signal control system, and also difficult to obtain. In numerous studies of turning proportion of traffic flows at intersections, a variety of models have been proposed to derive dynamic O-D matrices over the last

decades, such as Kalman filtering, recursive estimation, and so on. For most of the methods and models, however, it is difficult to meet the requirement which is the nonlinear, real-time performance and adaptability of calculating the intersection turning proportion. The estimation of turning proportion of traffic flows should be done in a signal cycle. When time-series link flow data is input to the system, the turning movements' feature of nonlinear can be reflected.

There is an extensive literatures concerning turning proportion of traffic flows at intersections. He (1995) uses back-propagation neural network to estimate turning proportion of traffic flows through link flow data; Li (2007) uses BP neural network to calculate the turning proportion of traffic flows at intersection and the result is verified. This paper attempts to further some practical issues associated with intersection turning proportion of traffic flows for isolated signal control intersection.

A SINGLE INTERSECTION MODEL

Given that there is no turning restriction for the intersection that this paper discusses. Consider an intersection consisting of r approaches and s exits as follows:

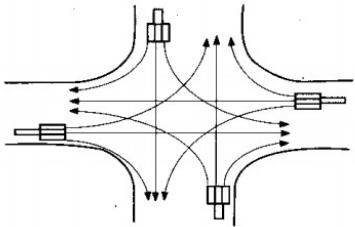


Figure 1. Relationship between Approaches and Exits

The following notation is used for the model formulation:
 $q_i(k)$, $i=1, 2, \dots, r$: the inflow counts from i approach during time interval k ;
 $y_j(k)$, $j=1, 2, \dots, s$: the outflow counts from j exit during time interval k ;
 $b_{ij}(k)$: the split parameter during time interval k .
And the relation between traffic flow and split parameter is

$$y_j(k) = \sum_{i=1}^r b_{ij}(k)q_i(k) \tag{1}$$

Considering the problems of general, it assumes that the intersection is of four phase signal control. The typical four phase signal control mode is shown in figure 2, the need of traffic flow data can be obtained by video image acquisition card.

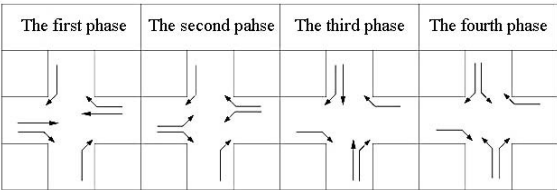


Figure 2. Intersection of Four Phase Signal Control

This article mainly aims at a single point of signal control intersection to seek a kind of accurate and efficient control method, and get the optimal signal cycle and time distribution of each phase. The ultimate goal is to make the traffic efficiency achieve optimal at the intersection.

THE B-P NEURAL MODEL ESTIMATING TURNING PROPORTIONS

Due to the following advantages when BPN is used in the intersection, this paper applies BPN to forecast the dynamic traffic flow turning proportions at intersection.

(1) BPN system has the very strong self-learning ability. And the hide layer can handle the complex prediction problem with the nonlinear output effect. The BPN can adapt to the short-term prediction of turning proportions which have the nonlinearity and severe changes in the intersection.

(2) BPNs are trained by presenting a series of samples repeatedly which were obtained by the traffic flow information detected by the detector in the approaches of the intersection. The turning proportions are predicted according the traffic flow information through the neural network self-learning and training methods as human.

(3)When the BPNs predict the turning proportions, the input is traffic flow in the upstream of the approaches in a signal cycle.

(4)The output is the turning proportions at the stop line in the approaches. (The U-turn vehicles are not considered.)

As the east import of an intersection an example, it has three lanes as the left, straight and right. So the BPNs are divided into three parts as the figure 3 when it is used to predict the turning proportions of the traffic flow.

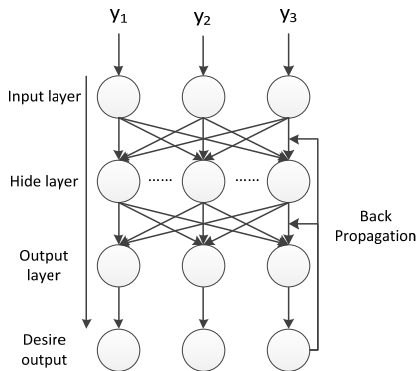


Figure 3. The Architecture of B-P Neural Network Predictor Model

Input layer: As shown in the figure 3, this layer has three neurons which are corresponded to the traffic flow of turning left, going straight and turning right in the east approach.

Hidden layer: It is advisable to take 15 neurons in this layer of the BPN used in the intersection through research.

Output layer: This layer has three neurons including east approach respectively in the direction of north, south and west these three turning proportions, a total of 3 output values.

The complete link structure is used in the whole BPN. The transfer function of hidden layer neurons employs the logarithmic sigmoid function. And the output values of the function are between 0 and 1 which is in conformity with the turning proportions range of the intersection.

The ordinary BPN has the disadvantages that the training speed is slow and it is hard to get the global optimal solution. However, the speed of network training can be enhanced greatly through combination of adaptive modified learning algorithm and batch gradient descent with momentum algorithm.

In the adaptive learning algorithm, the learning rate α is non-constant, it will change based on the last gradient as the following algorithm:

$$\omega(k+1) = \omega(k) + \alpha(k)D(k) \quad (2)$$

$$\alpha(k) = 2^\lambda \alpha(k-1) \quad (3)$$

$$\lambda = \text{sign}[D(k)D(k-1)] \quad (4)$$

Where the learning rate α is > 0 ; $\omega(k)$ is the weight for the k step; $D(k)$ is the gradient for the k step.

In the process of learning, the learning rate will double given that the sign of $D(k)$ is same as the last and the learning rate will halve if not.

The momentum gradient algorithm can solve the problem that the global optimization can't be obtained in the network training. The algorithm is as follows:

$$\omega(k+1) = \omega(k) + \alpha[(1-\varepsilon)D(k) + \varepsilon D(k-1)] \quad (5)$$

Where ε is the momentum factor and $0 < \varepsilon < 1$.

From the above two algorithms, the momentum-adaptive learning BPN weights modified algorithm is achieved as following:

$$\omega(k+1) = \omega(k) + \alpha[(1-\varepsilon)D(k) + \varepsilon D(k-1)] \quad (6)$$

The turning proportion of various approaches at the k period is obtained. And through the accumulation of b_{ij} for a certain time, the matrix of the turning proportion of various approaches B_{ij} is imported.

THE MULTI-OBJECTIVE TRAFFIC SIGNAL CONTROL MODEL FOR INTERSECTION

In order to achieve the purpose of real-time signal control, the signal control model for intersection based on BPN is established. According to the turning split parameter of BPN, the real-time traffic flow data is provided for signal timing. At the same time, considering the traffic flow status indicators such as the delay, queue length and effective capacity the nonlinear model for real-time signal timing of a single intersection is advanced. In this model, the objective function makes the vehicle delay, queue length and effective capacity as optimal as possible and the cycle length and split of each phase are configured real-time.

Due to the actual situation of road traffic in our country, especially in the rush hours, the interaction and interference among vehicles are more serious. So the weighted coefficient of performance indicators is added to the objective function and the weighted coefficient varies with the traffic flow of approaches, which can adapt to

timing requirements of different traffic conditions. The optimization model of the signal timing is established based on the traffic flow of each phase, and in the model the independent variable is the effective green time of each phase to minimize the objective function. The signal cycle and the green time are the key parameters of the signal timing, so the objective function makes the vehicle delay, queue length and effective capacity as optimal as possible. The nonlinear optimization model is as following:

$$\min f(G_x, T) = \sum_{x=1}^n [K_x^1 d_x + K_x^2 \bar{l}_x - K_x^3 Q_x] \quad (7)$$

$$\begin{cases} \sum_{x=1}^n (G_x + A_x + R_x) = T \\ 0.9p_x \leq \frac{G_x}{T} \leq 1.1p_x, \quad 1 \leq x \leq n \\ green_{x,min} \leq G_x \leq green_{x,max}, \quad 1 \leq x \leq n \\ \sum_{x=1}^n G_x + L \leq J, \quad 1 \leq x \leq n \\ G_x \geq 0, \quad 1 \leq x \leq n \end{cases} \quad (8)$$

where the three weighted coefficients are shown as:

$$K_x^1 = 2s_x p_x (1 - p) \quad (9)$$

$$K_x^2 = s_x p_x (1 - p) T \quad (10)$$

$$K_x^3 = 2(3600/T)p \quad (11)$$

G_x ($x = 1, 2, 3, 4$) is effective green light time of the first x phase; T is the signal cycle; d_x is average delay time of the vehicle arriving in the first x phase; $\bar{l}_x$ is the average queue length in the first x phase; Q_x is the effective traffic capacity of the approach x ; $green_{x,min}$ and $green_{x,max}$ are the minimum and maximum effective green time; n is the number of phase.

In the above model, while calculating the weighted coefficient K_x^1 , K_x^2 and K_x^3 the ratio of the traffic flow of the x phase, the saturated flow p_x and the sum of it at each phase P are obtained from dynamic split B_{ij} as the known data.

CASE STUDY

The intersection of Road Zhaodengyu and Ping'anli West Street is surveyed manually. To guarantee the universality, the chosen intersection satisfies the intersection of no territorial limit conditions, and is also the standard four phase crossroads. The survey data mainly includes traffic flow, delay, queue length and saturation headway.

Number the east, south, west, north import as 1, 2, 3, 4 separately. The turning proportion at every 2 minutes will be got by using the data surveyed in the 7:00 ~ 10:00 the morning rush hours. Thus there's a total of 90 periods of observation data.

As B_{14} , B_{31} , B_{34} are the east import right turn, west import going straight, west left turn proportion an example, the comparison line charts of the turning proportion value generated by BPN through Matlab and the real value are get as the following figures 4-6. And from the figures, the turning proportion value generated by BPN maintains the real value of ups and downs.

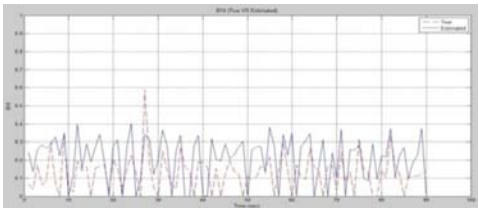


Figure 4. Comparison between Actual and Estimated Right Turning Proportion at East Approach

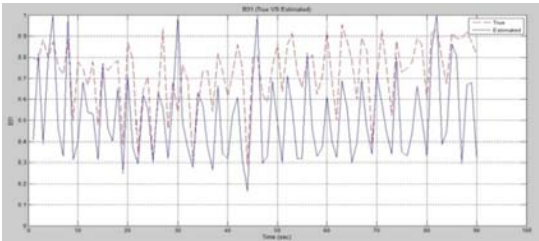


Figure 5. Comparison between Actual and Estimated Straight Proportion at West Approach

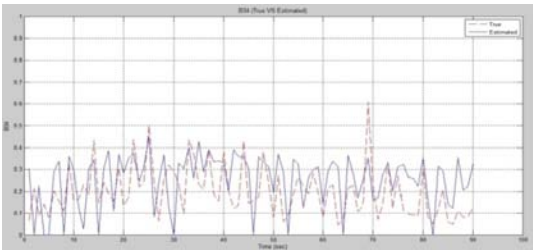


Figure 6. Comparison between Actual and Estimated Left Turning Proportion at West Approach

Mean square errors of the turning proportion value are divided into changes between 0.1-0.3. And the relative error is only 0.01 after the plus or minus cancellation of average relative error.

In order to reflect the effectiveness of the signal timing parameters generate by the model proposed in this article, the contrast of the Webster and the model proposed in this article is achieved at vehicle delay, queue length and effective capacity by using ten periods' traffic data as following Figure 7 to Figure 9:

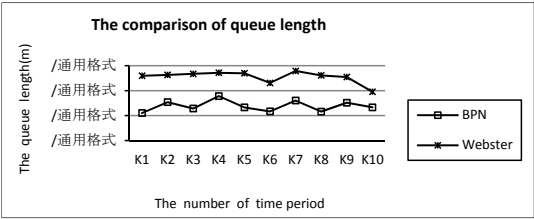


Figure 7. Comparison between BPN and Webster for Queue Length

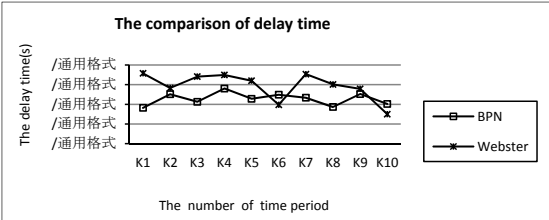


Figure 8. Comparison between BPN and Webster for Delay Time

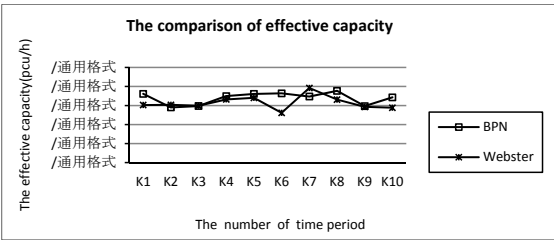


Figure 9. Comparison between BPN and Webster for Effective Capacity

Through the contrast, it shows that the signal control model based on the BPN is obviously better than the Webster algorithm in calculating the vehicle delay, queue length and effective capacity.

CONCLUSIONS

This article studies the signal control model based on BPN. The neural network technology is applied to forecast the turning proportion of the traffic flow in the intersection. And then the BPN model used for estimating the turning proportion is established which provides key data to the signal control model. Taking typical intersection and four-phase traffic signal controlling as an example, traffic flow datum in different periods were chosen for simulation. Finally, through the contrast with traditional Webster method, it shows that the improved signal control system based on BPN can meet the requirements of online control signal in the real time.

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The Tom Lantos Tunnels at Devil's Slide Project, the Longest Highway Tunnels in California

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ABSTRACT

The Tom Lantos Tunnels at Devil's Slide are located along the California coast between Pacifica and Montara, west of the San Francisco Bay Area in a region with steep and unstable geological formation. The project replaces the stretch of coastline for much of the distance between the two towns that had a long history of closure due to recurring rockslides. After years of studies, it was decided to bypass it with two inland tunnels, providing a safe, dependable highway between the two towns. The twin tunnels are 9m wide by 6.8m high and 1.3km long and sit in very complex geological settings in a high seismic zone and an environmentally-sensitive area. The tunnels were constructed beneath San Pedro Mountain using the New Austrian Tunneling Method—a method determined to be the most viable because of its flexibility, adaptability, and ability for adjustment locally to the site's changing ground conditions. At 1,310m long, it is the longest highway tunnel in California. (The Wawona Tunnel, completed during 1933 in Yosemite National Park, at 1,290m in length previously was the longest highway tunnel in California.)

INTRODUCTION

The Tom Lantos Tunnels at Devil's Slide are located along the California coast between Pacifica and Half Moon Bay, west of the San Francisco Bay Area in a region with steep and unstable geological formation. The project replaces the infamous coastal stretch of California's State Highway 1, which hugged the coastline for much of the distance between the two towns and had a long history of closure due to recurring rockslides. After years of study—and a rare popular referendum to consider a tunnel alternative over a major highway development—Caltrans decided to bypass the Devil's Slide with two inland tunnels, providing a safe, dependable highway between the two towns. The twin tunnels rest in one of the most complex geological settings in California. The alignment proceeds south from Pacifica, departs from existing SR 1 along a seven-percent uphill grade, crosses a valley at Shamrock Ranch, passes through a small ravine, enters the horseshoe-shaped tunnels 650 feet below San Pedro Mountain, and exits the tunnels just south of the Devil's Slide area where it rejoins SR 1.

The tunnels were constructed beneath San Pedro Mountain using the New Austrian Tunneling Method (NATM)—a method determined to be the most viable because of its ability to adjust to the site's changing ground conditions. Construction

began in 2007, and the tunnels opened to traffic on March 26, 2013, when they were officially named the Tom Lantos Tunnels, in honor of the late U.S. Senator who secured emergency federal funding for the project. Marking the first highway tunnels constructed in California since 1964, the twin tunnels are 9m wide by 6.8m high and 1.3km long. Contract bids for the project were opened in Sacramento on November 14, 2006. Kiewit Pacific submitted the winning bid of \$272,366,000 with a 1,500-day construction schedule.

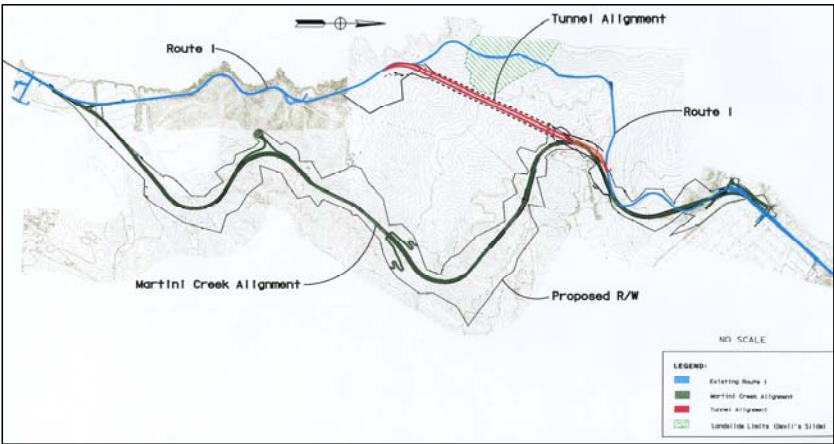


Figure 1. Tunnel alignment

PROJECT DESCRIPTION

The Tom Lantos Tunnels at Devil’s Slide consist of a separated two-lane road, one lane in each direction, which passes through twin tunnels and over twin bridges and then connects with the existing non-separated, two-lane road at each end. The length of the entire project is approximate 1,900 meters, made up of four major project sections and moving from south to north:

- 1) The Operations and Maintenance Center (OMC) area, the 250m-long south approach roadways, including the South Rock Cut (tall tie-back retaining walls), extends to the tunnels’ South Portals.
- 2) The twin tunnels are 1,310m long and extend to the North Portals near the south abutments of the twin bridges.
- 3) Twin segmental concrete box girder bridges are 275m and 300m long, respectively, and span the Shamrock Ranch valley.
- 4) North approach roadways then rejoin the existing highway.

The tunnels include the following features:

- Cut-and-cover portals
- Two 4ft emergency and maintenance walkways on either side
- Single 12ft traffic lane with an 8ft right shoulder and 2ft left shoulder

- 10 cross-passages for emergency egress
- Three equipment chambers
- State-of-the-art safety and communications system
- State-of-the-art fire detection and suppression system
- Carbon monoxide and nitrogen oxide detection system
- Electronic message signs displaying safety/advisory information
- Eight pairs of jet fans for ventilation
- High-quality lighting
- Operations control system
- Off-site operations and maintenance center
- Design that can withstand a 7.5 magnitude seismic event
- Bicycle accessibility

INNOVATIONS

The Tom Lantos Tunnels at Devil's Slide project employed several innovations.

State's first use of NATM. The length of the tunnels and the challenging ground conditions did not lend themselves to a tunnel boring machine technique. Instead, Caltrans chose to use the New Austrian Tunneling Method (NATM), a sequential excavation and support tunnel-mining technique. The tunnels are the first to be built in California using the NATM. This flexible, adaptable method relies on observation of ground behavior during excavation, which facilitated unprecedented in-field team collaboration. Based on instrumentation and monitoring of the rock deformation, geologists and engineers adjusted the rock supports "on the fly." This allowed the tunnel to be constructed safely, timely, and at a reasonable cost.

NATM incorporates the strength of the surrounding rock during construction and uses a double-shell support system. The system consists of an initial lining of sprayed fiber-reinforced shotcrete, rockbolts, and lattice girders. Once a section was excavated, crews installed lattice girders and then quickly applied an initial 6in. layer of shotcrete. The shotcrete included structural synthetic fiber to meet specified deflection requirements, a novel approach in the United States. The final lining, designed to accommodate seismic forces, was constructed from reinforced concrete. It was applied after rock dowels and other pre-support measures were implemented. The initial support and final lining are separated by a waterproofing and tunnel drainage system.

Tunnel excavation was accomplished using top heading, bench, and invert. Both tubes were excavated 60ft apart and simultaneously, beginning at the southern portals. Each time crews encountered a different rock type, they had to swap out an entire set of machinery. To improve excavation efficiency, the bench was excavated the full length of the reach prior to the installation of the invert.

Unprecedented in-field collaboration. The flexible, adaptable NATM relies on observation of ground behavior during excavation, which facilitated unprecedented in-field team collaboration. Specifically, engineers in the field—representing the owner, the designer, and the contractor—observed, with geologists, the excavation as

it happened and made modifications to the construction process “on the fly” based on the observed behaviors of the ground and rock. This allowed the tunnel to be constructed safely, timely, and at a reasonable cost.

Shotcrete lining with structural synthetic fiber, unique in the U.S. NATM incorporates the strength of the surrounding rock during construction and uses two types of support: an initial lining and a final lining. The initial shotcrete lining included structural synthetic fiber to meet specified deflection requirements, a novel approach in the United States. The final lining was made of traditional, cast-in-place reinforced concrete.

3-D modeling. In addition to two-dimensional numerical models of tunnel cross-sections, the project team prepared 3-D structural analysis models for a typical cross-passage and the transition of the tunnel to the north portal. Construction of the 10 cross-passages, which connect the two tubes, changes the rock stress field that impacts both tubes; therefore, to assess the impact of constructing the cross-passages on the stresses in the rock and, subsequently, the stresses in the tunnel liner, 3-D numerical modeling was required.

TECHNICAL CHALLENGES

The Tom Lantos Tunnels at Devil’s Slide were constructed in one of the most complex settings in California. The location presented four major technical challenges:

Technical Challenge 1: Variable rock formations. As part of a geotechnical baseline report, the project team collected rock data during the early stages of the design process by drilling many borings (up to 2,130m core length with horizontal bore up to 350m in length) along the proposed alignment. Laboratory tests performed on the rock cores provided engineers with the data necessary to identify and categorize 10 rock types that would be encountered during excavation.

The alignment contained highly-deformed Mesozoic granitic rock overlain by a series of thrust sheets, consisting of granitic rock and late-Cretaceous and early-Tertiary-age clastic sedimentary rock. The variety of rock was intermitted with hard and soft zones and consisted of blocky and seamy ground, squeezing rock and raveling ground. During the tunnel excavation, when tunnel excavation crews encountered softer soil types, they used articulated arms for digging. For harder rock, a 130-ton Voest Alpine ATM105 roadheader with rotating spike cylinders took over. Each time crews encountered a different rock type, they had to swap out an entire set of machinery. For the toughest areas, they drilled holes and used explosives.

Technical Challenge 2: High seismic zone. Because the tunnels lie along the active San Andreas Fault and four inactive fault lines, the subsurface structures comply with some of the toughest seismic criteria in the world. First, they had to withstand a 7.5–8.0 magnitude earthquake, the maximum movement geologists estimated for this arm of the San Andreas Fault. (The U.S. Geological Survey Working Group on California

Earthquake Probabilities has estimated a 21% probability of a strong earthquake—magnitude 6.7 or greater—occurring on the peninsula section of the San Andreas Fault within the next 30 years.)

Designed to accommodate seismic forces and allow for potential movements, the final lining of the tunnels eliminated expansion joints and consisted of two layers of reinforcement. The design team recommended two layers for the following reasons:

- 1) Key block loading occurs over much of the tunnels' length, which generally results in tensile stresses at the outside (rock) face of the final lining. Reinforcing steel was added to resist these tensile stresses. The design considered that the initial support would deteriorate over time and the final lining would be required to support all rock loadings, including key block loading.

- 2) Some areas of the tunnel produced unsymmetrical rock loadings. This loading necessitated two layers of reinforcement at these areas and areas of discontinuity, such as the tunnel and cross-passage junctions.

- 3) Final lining distortions from seismic ground displacements frequently caused tension stresses at the outside lining face under some combinations of lining self-weight and geostatic loading.

After a design seismic event, the tunnels are to remain serviceable and have experienced only "repairable" damage. "Serviceable" is defined as providing immediate access to emergency vehicles and full access to normal traffic almost immediately. "Repairable damage" is defined as allowing moderate inelastic response of the tunnel concrete liner and portals to occur. Concrete cracking, reinforcement yielding, and some spalling of cover concrete over the reinforcement is expected at this level. However, there will be no structural failure and damage will be limited to permit restoration of the structure (essentially the tunnel's pre-earthquake condition) without replacing any portion of the lining or portals. Damage will be repairable within 90 days and only by requiring lane closures during off-peak traffic periods.

Technical Challenge 3: Mixed ground conditions. The groundwater table lies above the tunnel alignment, and original projections showed that excessive amounts of groundwater were anticipated during construction. The construction sequence, starting from the south portal working north at a 2 percent slope, allowed the groundwater/formation water to flow by gravity to the south portal. Groundwater flows into the tunnel through the rock dowel boreholes and relief holes in the shotcrete lining, as well as through weep holes located near the longitudinal drain pipes. Dewatering ahead, using horizontal drainage holes drilled through the excavation face, was essential in areas of expected high water ingress. In the north block, crews encountered excessive water flow from a fault acting as a drain, collecting groundwater from the rock mass. To eliminate the water, three horizontal dewatering drain holes were drilled through the fault. One hole drained by gravity; the other two were inclined slightly and equipped with pumps. More than 10 million gallons of water were drained to keep the tunnel headings dry.

Technical Challenge 4: Environmentally-sensitive area. As if the rock formations, seismic activity, and ground conditions were not challenge enough, federally-

protected Peregrine falcons nested near the construction site, and California's endangered red-legged frogs were found living in the Shamrock Valley wetland area. The alignment of the tunnels was carefully chosen to respect these environmental concerns. To protect the endangered frog habitat, no equipment was allowed to enter the wetland area. Crews cordoned the area with frog fences and conducted extensive wetland monitoring to ensure the frogs' safety. To minimize the project's impact and to connect the north portal to SR 1 without disrupting the wetland area, twin balanced, cantilever, cast-in-place segmental concrete box girder bridges were constructed. They are the first bridges of their kind in California.

PROJECT RISK

Top risks specific to this project were local environmental concerns, potential contractor claims, and the unknown. Following were the strategies for mitigating each one.

Local environmental concerns. Knowing public opposition killed plans for a six-lane inland bypass, the local communities were proactively involved and regulatory permitting agencies were consulted in very early in the design and permitting process. Over the course of two years, numerous meetings were held with stakeholders to discuss aesthetics and other project elements. Through extensive collaboration, the context-sensitive design was developed that respects the community's concerns and is considerate of the world-class natural setting. These collaborative efforts led to a project that minimizes physical impacts to the existing pristine landscape by protecting vegetation, drainage courses, wetlands, existing landforms and open space. The project advanced with unprecedented community support.

Potential contractor claims. Caltrans desired to have a comprehensive bid package to lessen the likelihood of potential contractor claims, change orders, and cost increases. To mitigate those risks, world known tunnel expertise was retained to help to prepare plans, specifications and estimates for construction.

Unknown risks. World-known tunnel expertise was sought and peer review panels were held, identifying every possible risk that might encounter before, during, and after the project. These brainstorming resulted in an exhaustive risk register that groups the risks according to category: technical risks, external risks, organizational risks, project management risks, environmental risks, right-of-way risks, and construction risks. Each risk was then ranked according to the likelihood of occurrence, conditions that could trigger each event, outlined mitigation strategies, and the cost of impact to both schedule and budget, and oversight of each register entry was assigned to specific team members. Monthly and quarterly meetings were held to review the status of each risk and update the register.

SUCCESSFUL COMPLETION

A+B contracting. Caltrans led the project as a design-bid-build job in 2006 under a

cost-plus time bidding structure known as A+B contracting. Under this method, the time to complete the project (B) is assigned a monetary value and combined with the contract items-based bid (A). The bidder with the lowest overall combined bid (A+B) is awarded the contract.

Extensive public involvement. The project team had two objectives: establish a permanent connection between Pacifica and Half Moon Bay and meet environmental and local expectations. An extensive public involvement campaign was conducted that gathered residents' perspectives on everything from how the portals would look to how many lanes each bore would contain.

As a result of Caltrans' public outreach effort, its relationship with the residents of Pacifica and Half Moon Bay is much better than at the project's onset when residents, protective of their bucolic community, were skeptical of the project's ability to be sensitive to the environment. The same people who attended public hearings 13 years earlier also attended the ribbon-cutting ceremony and expressed how pleased they were with the tunnels' design, how well they blend with the surrounding mountainside, and the respect for the environment.

High aesthetic value. Public input on how the tunnels would relate to their surroundings was vital to the project's success. To solicit opinions, Caltrans created an Aesthetics Committee that participated in a two-phased process to address key aesthetic issues that would impact the tunnels' designs. Committee members included two representatives from the Pacifica City Council, two non-participating representatives from the Half Moon Bay City Council, two representatives from the Midcoast Community Council, one representative from the San Mateo County Board of Supervisors, Caltrans staff, and consultants.

The committee met bimonthly from August 2001 to March 2003. Its goal was to develop project concepts that conform to the context of the Devil's Slide environment; minimize physical impacts to existing landscape elements, including vegetation, drainage courses, and wetlands; and minimize visual impacts to existing landforms and open spaces. The Aesthetics Committee documented its recommendations in a report and presented the information to its constituents at the end of each phase.

Tunnel portals were designed to meet context-sensitive aesthetic and safety requirements and were approved by both the California Coastal Commission and local commissions. Each end of the tunnels is different. The north portals' architectural texture matches the north mountain terrain, and the south portals blend with the south mountain terrain. In total, 20 different design concepts were created before the Coastal Commission approved what exists today.

Each tunnel has a vertical clearance of 15.5ft. The portal entrances have a 22-degree vertical skew, and the north portal entrances have a 57-degree horizontal skew. Each entrance has a bull nose composed of up to 21 different elliptical shapes gradually transforming from one end to the other. The skews and the elliptical shapes required complicated 3-D modeling for fabricating forms and rebar and installing the forms and rebar. Faux boulders crafted by the artist who designed Disneyland's Indiana Jones ride were strategically and safely positioned over the portal entrances.

In addition to their aesthetic function, the boulders act as deflective shields, protecting vehicles from rock slides. Rock bolting and permanent steel mesh netting also provide rockfall protection at the south portals.

Caltrans permanently closed the dangerous stretch of roadway after the tunnels opened. It now is a pedestrian and bicycle trail, offering some of the most spectacular vistas in the United States

CONCLUSIONS

The new \$439 million Tom Lantos Tunnels are among the safest in the nation, meeting every requirement of the National Fire Protection Association's 502 Code and the Americans with Disabilities Act.

This project is an excellent example of an engineering team's commitment to identifying and evaluating all aspects of a project and developing a solution through a cooperative partnership with all involved parties and agencies. The work conducted was geologic and geotechnical site characterization, portal stability analysis, geohydrologic modeling, and soil structure interaction for the tunnel support system loads. It also included the design of the tunnel lining and portal; initial ground support using the New Austrian Tunnel Method construction; final lining design, including seismic analyses; soil-nail retaining walls; ventilation design using jet fans; and electrical, lighting, and systems monitoring design.

The constructed project has significantly improved safety on the infamous coastal stretch of California's State Highway 1 and provided a structure that fits aesthetically with the area.

ACKNOWLEDGMENTS

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A Study of Nonlinear Time History Analysis vs. Current Codes Analysis Procedure of Comparing Linear Dynamic Demand with Nonlinear Static Capacity for Ordinary Standard Bridge

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ABSTRACT

Nonlinear time history analysis is known simulating a structure behavior under severe earthquake more proper than other methods. However for simplicity, most of bridges in category of Ordinary Standard Bridge (OSB) are being analyzed by a combined procedure which consists of a linear ARS analysis for earthquake response (demand) and a static nonlinear pushover for ultimate displacement (capacity) per guideline of many transportation agencies worldwide. The demand and capacity are then compared to determine the safety of the bridge. For single degree freedom (SDF) system this procedure has been proven an effective method with satisfactory accuracy. For bridges in the category of OSB but with noticeable characteristics of MDF (multi-degree of freedom) system, large discrepancies between deformation patterns from linear analysis and nonlinear pushover are often observed by engineers, so the accuracy of conclusion from this procedure is questioned. To explore nonlinear dynamic behavior of these bridges and investigate the adequacy of the popular combined linear with nonlinear analysis procedure, a series bridges within category of OSB ranging from slight to severe mass and stiffness unbalance were analyzed. The analysis methods used for each bridge include linear and nonlinear time history analysis, linear ARS analysis and nonlinear static pushover. To ensure valid results comparison, a ground acceleration time history is used for both linear and nonlinear time history analysis, its frequency domain ARS curve is used for ARS analysis. Selected bridges model, ground acceleration and analysis methods, procedures, results, comparison, discussion, conclusions and suggestions are all presented in the paper.

INTRODUCTION

Due to the complicity and time consuming of bridge nonlinear time history analysis, most of bridges in OSB category are currently designed based on a simplified seismic analysis procedure which consists of a linear ARS analysis for earthquake response (demand) and a static nonlinear pushover for ultimate displacement (capacity) per guideline of many transportation agencies worldwide. The demand from elastic analysis is compared to the capacity from nonlinear analysis to determine the safety of the bridge. This simplified procedure is an extended application of a proven method for single degree freedom (SDF) system. The accuracy of this simplified

procedure is often questioned when engineers are working on the bridge which possesses characteristics of MDF (multi-degree of freedom) system even they do belong to category of OSB, because noticeable discrepancies between deform pattern from linear analysis and nonlinear pushover are observed. To explore nonlinear dynamic behavior of these bridges and investigate the adequacy of the combined linear with nonlinear analysis procedure, a series bridges within category of OSB ranging from symmetric, slight to severe mass and stiffness unbalance are analyzed. The analysis methods used for each bridge include linear and nonlinear time history analysis, linear ARS analysis and nonlinear static pushover. A selected ground acceleration time history is used for both linear and nonlinear time history analysis, its frequency domain ARS curve is used for ARS analysis. Two single column models subjected to earthquake inputs of different strengths are also analyzed to show how well the aforementioned simplified procedure works for SDF system and how it is changed in MDF system. This paper presents the investigation analysis which includes selected bridges model, ground acceleration, analysis methods, procedures, results, comparison, discussion, conclusions and suggestions.

SAMPLE STRUCTURES AND ANALYSIS MODELS

Analyzed Bridges and Systems. The investigation and analysis include total four bridge structures and two single columns, namely Bridge-1, Bridge-2, Bridge-3, Bridge-4, Column-A and Column-B. All bridges and columns are concrete structures. The table below listed the geometry and type of the analyzed bridges.

Table 1. Geometry Data and Type of the Analyzed Bridges								
Name	# of Span	Bridge Length (ft)	Bridge Width (ft)	Span Length (ft)	Skewed Angle (degree)	Column Length	Superstructure type	Substructure type
Bridge-1	3	202.30	27'-8"	60.53	A1: 0	-	Concrete precast P/S I-Girder with CIP Deck	5'-0' column on pile group, fix-fix joints. Seat type abutment
				81.24	Bnts: 0	B2: 26'-5"		
				60.53	A4: 0	B3: 27'-1"		
Bridge-2	3	202.30	27'-8"	60.53	A1: 45	-	Concrete precast P/S I-Girder with CIP Deck	5'-0' column on pile group, fix-fix joints. Seat type abutment
				81.24	Bnts: 45	B2: 26'-5"		
				60.53	A4: 45	B3: 27'-1"		
Bridge-3	3	214.70	33'-3"	70.00	A1: 19	-	Concrete precast P/S I-Girder with CIP Deck	5'-0' column on pile group, fix-fix joints. Seat type abutment
				82.00	Bnts: 19	B2: 26'-0"		
				62.70	A4: -23	B3: 26'-0"		
Bridge-4	6	540.78	42' at Abut-1 and taped to 15'	67.54	-	-	Concrete precast P/S I-Girder with CIP Deck	6'-0" column on 8'-0" CIDH pile shaft. Seat type abutment
				97.92	-	B2: 64'-3"		
				97.92	-	B3: 67'-3"		
				97.80	A1: 25.05	B4: 70'-2"		
				105.73	Bnts: 25.05	B5: 72'-5"		
				73.88	A7: 25.05	B6: 74'-8"		

Note: A1 is Abutment 1, B2 is Bent 2, and so on.

It can be seen from the above table that Bridge-1 is a symmetric structure system. The skewed angle and un-uniformity increased from Bridge-2 to Bridge-3 and Bridge-4 is the most irregular one among all four bridges.

Column-A and Column-B both are 5'-0" diameter reinforced concrete cantilevered columns with a clear depth of 21'-6". Column-A supports a concentrated weight of 600 kips at top of the column to represent superstructure while Column-B supports 750 kips. Both columns represent SDF systems with different natural frequency due to difference of the supporting mass.

The computer models of all the analyzed bridges are shown below in Figure 1 through Figure 4.

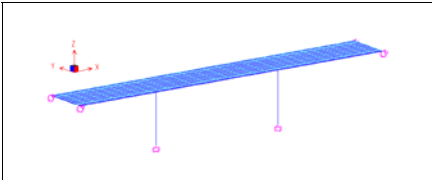


Figure 1. Computer Model of Bridge-1

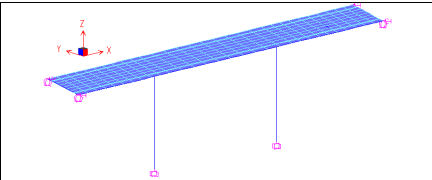


Figure 2. Computer Model of Bridge-2

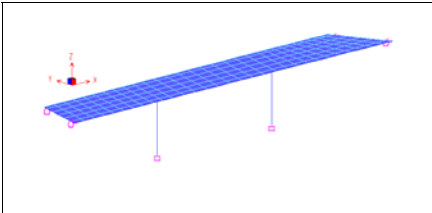


Figure 3. Computer Model of Bridge-3

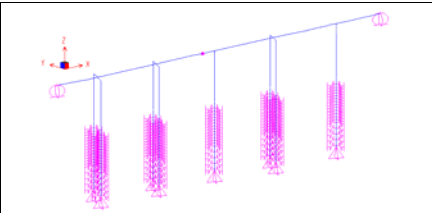


Figure 4. Computer Model of Bridge-4

Ground Motions. Imperial Valley earthquake record IMPVALL/H-E07239 with 5% damping and 0.463g PGA recorded on October 1979 is selected as seismic excitation for the study. The 30 seconds acceleration record is used in both linear elastic time history analysis and nonlinear time history analysis. The frequency domain ARS curve of the record is also used for ARS analysis of all bridge models for obtaining demand displacements as specified in most of the bridge seismic design guidelines. Kobe earthquake record of year 1995, KOBE/KJM000 with 5% damping and 0.821g PGA is also applied as input to some models for both linear and nonlinear analysis to see the sensitivity of the structure to different ground motions and to have better universality of the conclusions drawn from the study. The ground motions are from PEER (Pacific Earthquake Engineering Research Center) with headquarters at University of California, Berkeley.

To investigate bridge seismic response in different post yielding stages, Imperial Valley earthquake record is multiplied by a series of constants to create several seismic excitations with different strengths. In order to clearly identify the structural 3-D response effect, single direction seismic input is used in each analysis

case. The acceleration records and their frequency domain ARS curves are shown in Figure 5 and Figure 6.

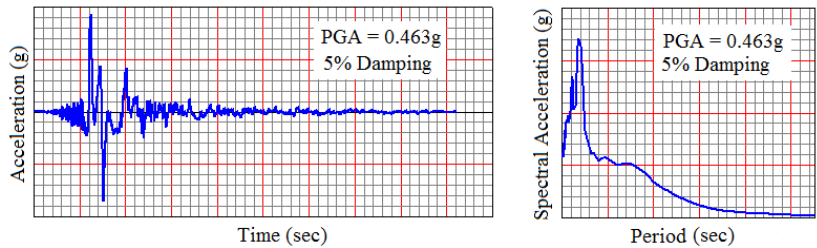


Figure 5. Acceleration Record and ARS Curve of Imperial Valley Earthquake, 1979

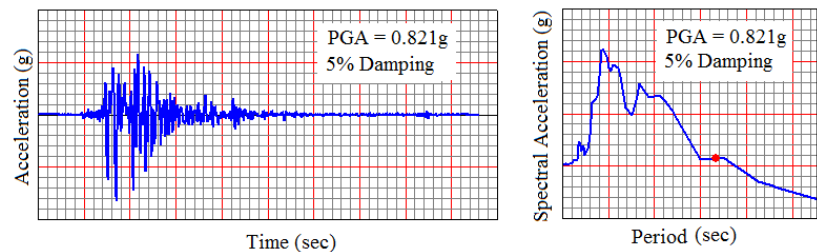


Figure 6. Acceleration Record and ARS Curve of Kobe Earthquake, 1995

Analysis Method and Analysis Cases. For each bridge model elastic ARS analysis, static nonlinear pushover analysis, linear time history analysis, and nonlinear time history analysis are carried out. In summary, total 140 cases’ analysis are carried out for this study with the computer program of SAP2000. The detail models and cases are listed in the Table 2 through Table 4 below for reference.

Table-2. Summary of Analysis Cases for SDF System (Single Columns)				
Ground Motion: Imperial Valley, 1979			Ground Motion: Kobe, 1995	
Excitation	Cloumn-A	Cloumn-B	Excitation	Cloumn-A
Muliplier	Analysis Case	Analysis Case	Muliplier	Analysis Case
1.00	ARS	ARS	0.50	ARS
	Static Pushover	Static Pushover		Static Pushover
	Linear Time History	Linear Time History		Linear Time History
	Nonlinear Time History	Nonlinear Time History		Nonlinear Time History
1.25	ARS	ARS	0.75	ARS
	Static Pushover	Static Pushover		Static Pushover
	Linear Time History	Linear Time History		Linear Time History
	Nonlinear Time History	Nonlinear Time History		Nonlinear Time History
1.50	ARS	ARS	1.00	ARS
	Static Pushover	Static Pushover		Static Pushover
	Linear Time History	Linear Time History		Linear Time History
	Nonlinear Time History	Nonlinear Time History		Nonlinear Time History

Table 3. Summary of Analysis Cases for Bridge-1 & Bridge-2

Ground Motion	Incidence Direction	Bridge-1		Bridge-2	
		Excitation Multiplier	Analysis Case	Excitation Multiplier	Analysis Case
Imperial Valley, 1979	L-Dir	1.00	ARS	1.00	ARS
			Static Pushover		Static Pushover
			Linear Time History		Linear Time History
			Nonlinear Time History		Nonlinear Time History
	T-Dir		ARS		ARS
			Static Pushover		Static Pushover
			Linear Time History		Linear Time History
			Nonlinear Time History		Nonlinear Time History
	L-Dir	1.10	ARS	1.10	ARS
			Static Pushover		Static Pushover
			Linear Time History		Linear Time History
			Nonlinear Time History		Nonlinear Time History
	T-Dir		ARS		ARS
			Static Pushover		Static Pushover
			Linear Time History		Linear Time History
			Nonlinear Time History		Nonlinear Time History
	L-Dir	1.20	ARS	1.20	ARS
			Static Pushover		Static Pushover
			Linear Time History		Linear Time History
			Nonlinear Time History		Nonlinear Time History
	T-Dir		ARS		ARS
			Static Pushover		Static Pushover
			Linear Time History		Linear Time History
			Nonlinear Time History		Nonlinear Time History
Kobe, 1995	L-Dir	1.00	ARS		-
			Static Pushover		
			Linear Time History		
			Nonlinear Time History		
	T-Dir		ARS		
			Static Pushover		
			Linear Time History		
			Nonlinear Time History		

For both linear and nonlinear time history analysis, direct integration with Hilber-Hughes-Taylor method is used. Rayleigh damping with coefficients defined by structure nature periods is applied. Nonlinear P-Δ effect is also included in the nonlinear time history analysis and the analysis starts after the structure dead load is fully loaded. For static nonlinear pushover, two lateral loading cases are pushed for each model. One is associated with uniform lateral gravity loading and the other is a set of concentrated force at top of each column which equals to column elastic shear force from the bridge’s ARS analysis. Both sets of lateral load have very closed pushover results. All analyzed bridges are detailed to have nonlinear plastic hinges concentrated at substructure columns, and abutment shear keys are designed to be

fused when bridge subjects to design earthquake per current AASHTO and Caltrans seismic design criteria. Therefore in the models the structure nonlinear plastic components are column plastic hinges represented by PMM yielding surface at top and bottom of the columns as well as the shear key residual resistance represented by lateral nonlinear links. The shear key residual elastic stiffness and maximum resistance is assigned a value of about 10% of the adjacent column bent to prevent girder’s swinging effect per code. The soil passive resistance at abutment is ignored for better observation of the substructure columns’ dynamic behavior. All bridges in the study belong to OSB category bridges. The 1st natural period of each bridge is greater than 0.7sec and the difference of stiffness/mass ratio of adjacent bent is within 25%. The difference of this ratio of any two bents in a frame is less than 50%.

Table 4. Summary of Analysis Cases for Bridge-3 & Bridge-4					
Ground Motion	Incidence Direction	Bridge-3		Bridge-4	
		Excitation Multiplier	Analysis Case	Excitation Multiplier	Analysis Case
Imperial Valley, 1979	L-Dir	1.00	ARS	1.00	ARS
			Static Pushover		Static Pushover
			Linear Time History		Linear Time History
			Nonlinear Time History		Nonlinear Time History
	T-Dir		ARS		ARS
			Static Pushover		Static Pushover
			Linear Time History		Linear Time History
			Nonlinear Time History		Nonlinear Time History
	L-Dir	1.25	ARS	1.25	ARS
			Static Pushover		Static Pushover
			Linear Time History		Linear Time History
			Nonlinear Time History		Nonlinear Time History
	T-Dir		ARS		ARS
			Static Pushover		Static Pushover
			Linear Time History		Linear Time History
			Nonlinear Time History		Nonlinear Time History
	L-Dir	1.50	ARS	1.50	ARS
			Static Pushover		Static Pushover
			Linear Time History		Linear Time History
			Nonlinear Time History		Nonlinear Time History
	T-Dir		ARS		ARS
			Static Pushover		Static Pushover
			Linear Time History		Linear Time History
			Nonlinear Time History		Nonlinear Time History

ANALYSIS RESULTS AND DISCUSSION

SDF Systems. Table 5 below listed the displacements of Column-A and Column-B from ARS analysis, LTH (linear time history) analysis, NLTH (nonlinear time history) analysis and static nonlinear pushover analysis. It can be seen that the ratios of the maximum displacement of ARS analysis to LTH analysis are about 1.0. This is

expected because both are linear dynamic response to the same earthquake record. The ratios of the maximum displacement of LTH analysis to NLTH analysis are also very close to 1.0 disregarding the change of the ground motion and structure. This result re-confirms that the popular simplified seismic analysis procedure which treating the displacement from linear elastic dynamic analysis as real structure displacement response including nonlinear behavior works great for SDF. However a few cases among all 12 analyzed cases the NLTH displacement is smaller than LTH displacement with a difference about 35%. These special cases require more study.

Table 5. Top of Column Displacement of SDF Sytem Models

Analysis Method	Ground Motion: Imperial Valley, 1979						Ground Motion: Kobe, 1995		
	Column-A			Column-B			Column-A		
	1st Mode T = 0.8094 sec			1st Mode T = 0.9036 sec			1st Mode T = 0.8094 sec		
	1.0EQ	1.25EQ	1.5EQ	1.0EQ	1.25EQ	1.5EQ	0.5EQ	0.75EQ	1.0EQ
	Δ_{max} (ft)	Δ_{max} (ft)	Δ_{max} (ft)	Δ_{max} (ft)	Δ_{max} (ft)	Δ_{max} (ft)	Δ_{max} (ft)	Δ_{max} (ft)	Δ_{max} (ft)
Yield	0.13	0.13	0.13	0.14	0.14	0.14	0.13	0.13	0.13
Static NL Pushover	1.21	1.21	1.21	1.19	1.19	1.19	1.21	1.21	1.21
ARS	0.70	0.87	1.05	0.67	0.84	1.01	0.51	0.76	1.01
LTH	0.70	0.88	1.05	0.67	0.83	1.00	0.48	0.73	0.97
NLTH	0.63	0.85	1.08	0.63	0.86	1.15	0.46	0.73	0.89
Ratio of ARS/LTH	1.00	1.00	1.00	1.01	1.01	1.01	1.04	1.04	1.04
Ratio of NLTH/LTH	0.90	0.97	1.03	0.94	1.03	1.15	0.96	1.00	0.92
Ratio of NLTH/Push	0.52	0.70	0.89	0.53	0.72	0.97	0.38	0.60	0.73
Ratio of NLTH/Yield	4.69	6.33	8.07	4.54	6.19	8.26	3.46	5.45	6.64
Column Status	Stable	Failed	Failed	Stable	Failed	Failed	Stable	Failed	Failed

Note: Δ_{max} : Maximum displacement at top of the column.

Yield: 1st yielding displacement of the system

NL Pushover (or Push) is nonlinear static pushover analysis

ARS: Acceleration response spectrum analysis

LTH: Linear time history analysis

NLTH: Nonlinear time history analysis

An interesting observation comes from the ratios of the max NLTH displacement to the column yielding displacement (ductility demand) and the ratios of NLTH displacement to capacity displacement from nonlinear static pushover (D/C ratio of the displacement). It is seen that in the cases analyzed, when the ductility demand greater than 5 the column status is failed, while the D/C ratio of the displacement is less than 1, some are even far less than 1. This observation tells that simply limiting the demand displacement to be less than static capacity displacement may not be enough to ensure the safety of the structure. Dynamic capacity displacement can be much smaller than static capacity displacement therefore limiting the demand ductility to certain number is important when using simplified analysis

procedure for design. Caltrans’ SDC (Seismic Design Criteria) sets a limit of 4 for single column bents supported on fixed foundation. The number came out from this study (< 5) shows it is an adequate threshold.

Bridge Models. Table 6 through Table 9 below listed the analysis results for Bridge-1, Bridge-2, Bridge-3 and Bridge-4.

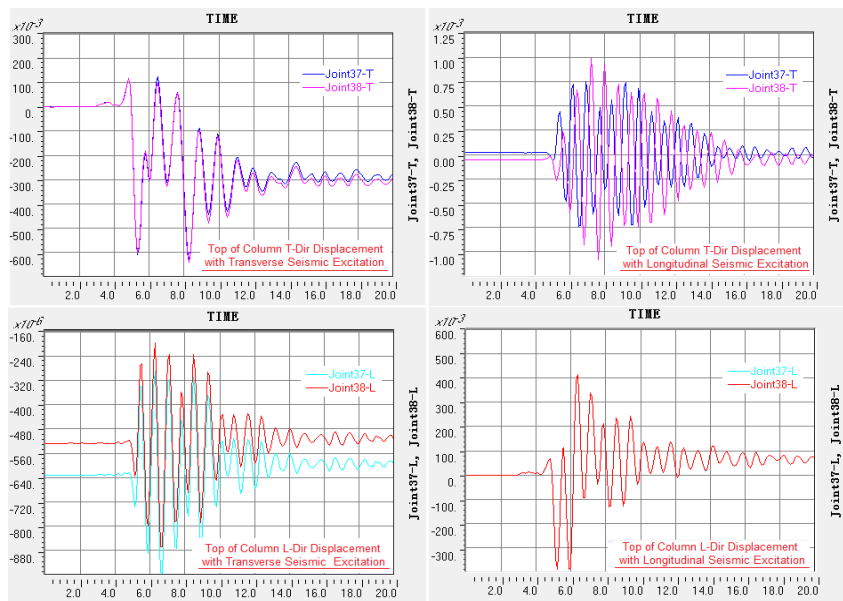
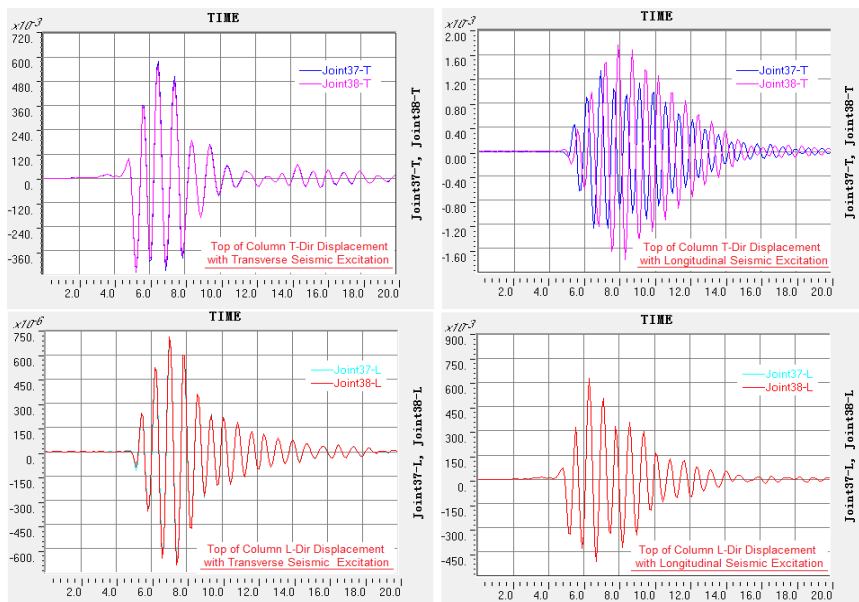
Table 6. Top of Column Displacement of Bridge-1							(T1 = 0.8094 sec)					
Analysis Method	Ground Motion: Imperial Valley, 1979											
	T-Dir Displacement with T-Dir Incidence						L-Dir Displacement with L-Dir Incidence					
	1.0EQ		1.1EQ		1.2EQ		1.0EQ		1.1EQ		1.2EQ	
	B2 Δmax	B3 Δmax	B2 Δmax	B3 Δmax	B2 Δmax	B3 Δmax	B2 Δmax	B3 Δmax	B2 Δmax	B3 Δmax	B2 Δmax	B3 Δmax
	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)
Yield	0.26	0.28	0.26	0.28	0.26	0.28	0.17	0.17	0.17	0.17	0.17	0.17
NL Pushover	1.24	1.57	1.24	1.57	1.24	1.57	1.21	1.25	1.21	1.25	1.21	1.25
ARS	0.58	0.60	0.64	0.66	0.70	0.71	0.62	0.62	0.68	0.68	0.75	0.75
LTH	0.58	0.59	0.64	0.65	0.69	0.71	0.63	0.63	0.69	0.69	0.75	0.75
NLTH	0.64	0.63	0.69	0.69	0.76	0.77	0.43	0.43	0.45	0.45	0.50	0.50
ARS/LTH	1.00	1.00	1.00	1.00	1.00	1.00	1.01	1.01	1.01	1.01	1.01	1.01
NLTH/LTH	1.11	1.06	1.09	1.05	1.10	1.08	0.68	0.68	0.65	0.65	0.66	0.66
NLTH/Push	0.52	0.40	0.56	0.44	0.61	0.49	0.35	0.34	0.37	0.36	0.41	0.40
NLTH/Yield	2.44	2.25	2.63	2.45	2.89	2.74	2.45	2.45	2.58	2.59	2.86	2.86
ARS/Push	0.47	0.38	0.51	0.42	0.56	0.46	0.51	0.50	0.57	0.55	0.62	0.60
ARS/Yield	2.21	2.13	2.43	2.34	2.65	2.55	3.58	3.59	3.94	3.95	4.30	4.31
Column Status	Stable	Stable	Stable	Stable	Stable	Stable	Stable	Stable	Stable	Stable	Stable	Stable
Ductility Capacity = Static NL Pushover/Yield =						5.61 (T-Dir)			6.97 (L-Dir)			

Table 7. Top of Column Displacement of Bridge-2							(T1 = 0.9638 sec)					
Analysis Method	Ground Motion: Imperial Valley, 1979											
	T-Dir Displacement with T-Dir Incidence						L-Dir Displacement with L-Dir Incidence					
	1.0EQ		1.25EQ		1.5EQ		1.0EQ		1.25EQ		1.5EQ	
	B2	B3	B2	B3	B2	B3	B2	B3	B2	B3	B2	B3
	Δmax	Δmax	Δmax	Δmax	Δmax	Δmax	Δmax	Δmax	Δmax	Δmax	Δmax	Δmax
	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)
Yield	0.28	0.30	0.28	0.30	0.28	0.30	0.16	0.16	0.16	0.16	0.16	0.16
NL Pushover	1.53	1.58	1.53	1.58	1.53	1.58	1.01	1.41	1.01	1.41	1.01	1.41
ARS	0.54	0.55	0.67	0.68	0.77	0.79	0.62	0.62	0.77	0.77	0.90	0.90
LTH	0.53	0.55	0.65	0.68	0.76	0.78	0.62	0.62	0.78	0.78	0.90	0.90
NLTH	0.67	0.63	0.83	0.81	0.99	0.97	0.43	0.43	0.65	0.65	0.79	0.79
ARS/LTH	1.02	0.99	0.98	0.99	0.98	0.99	1.01	1.01	1.00	1.00	1.00	1.00
NLTH/LTH	1.26	1.15	1.27	1.20	1.31	1.24	0.69	0.69	0.84	0.84	0.88	0.88
NLTH/Push	0.44	0.40	0.54	0.51	0.65	0.62	0.42	0.30	0.64	0.46	0.78	0.56
NLTH/Yield	2.43	2.13	3.02	2.74	3.61	3.30	2.65	2.64	4.02	4.00	4.89	4.87
ARS/Push	0.35	0.35	0.44	0.43	0.50	0.50	0.61	0.44	0.76	0.55	0.89	0.64
ARS/Yield	1.96	1.87	2.42	2.31	2.81	2.68	3.84	3.82	4.79	4.76	5.55	5.53
Column Status	Stable	Stable	Stable	Stable	Failed	Failed	Stable	Stable	Failed	Failed	Failed	Failed
Ductility Capacity = Static NL Pushover/Yield =						5.35 (T-Dir)			6.27 (L-Dir)			

Table 8. Top of Column Displacement of Bridge-3 (T1 = 1.0406 sec)												
Analysis Method	Ground Motion: Imperial Valley, 1979											
	T-Dir Displacement with T-Dir Incidence						L-Dir Displacement with L-Dir Incidence					
	1.0EQ		1.25EQ		1.5EQ		1.0EQ		1.25EQ		1.5EQ	
	B2 $\Delta_{\max}$ (ft)	B3 $\Delta_{\max}$ (ft)	B2 $\Delta_{\max}$ (ft)	B3 $\Delta_{\max}$ (ft)	B2 $\Delta_{\max}$ (ft)	B3 $\Delta_{\max}$ (ft)	B2 $\Delta_{\max}$ (ft)	B3 $\Delta_{\max}$ (ft)	B2 $\Delta_{\max}$ (ft)	B3 $\Delta_{\max}$ (ft)	B2 $\Delta_{\max}$ (ft)	B3 $\Delta_{\max}$ (ft)
Yield	0.26	0.26	0.26	0.26	0.26	0.26	0.17	0.17	0.17	0.17	0.17	0.17
NL Pushover	1.44	1.44	1.44	1.44	1.44	1.44	1.14	1.14	1.14	1.14	1.14	1.14
ARS	0.52	0.47	0.65	0.58	0.75	0.68	0.65	0.65	0.81	0.81	0.94	0.94
LTH	0.51	0.48	0.63	0.59	0.73	0.69	0.65	0.65	0.81	0.81	0.94	0.94
NLTH	0.65	0.66	0.85	0.81	1.02	0.97	0.44	0.44	0.58	0.58	0.69	0.69
ARS/LTH	1.02	1.02	0.98	1.02	0.98	1.02	1.00	1.00	1.00	1.00	1.00	1.00
NLTH/LTH	1.28	1.39	1.34	1.36	1.39	1.41	0.67	0.67	0.71	0.71	0.73	0.73
NLTH/Push	0.45	0.46	0.59	0.56	0.71	0.68	0.38	0.38	0.51	0.51	0.61	0.61
NLTH/Yield	2.46	2.57	3.22	3.13	3.86	3.78	2.55	2.55	3.41	3.41	4.05	4.05
ARS/Push	0.36	0.32	0.45	0.40	0.52	0.47	0.57	0.57	0.72	0.72	0.83	0.83
ARS/Yield	1.96	1.81	2.45	2.26	2.84	2.62	3.82	3.82	4.78	4.78	5.54	5.54
Column Status	Stable	Stable	Stable	Stable	Failed	Failed	Stable	Stable	Failed	Failed	Failed	Failed
Ductility Capacity = Static NL Pushover/Yield =						5.58 (T-Dir)			6.68 (L-Dir)			

Table 9. Top of Column Displacement of Bridge-4 (T1 = 2.8043 sec)										
Analysis Method	Ground Motion: Imperial Valley, 1979									
	T-Dir Displ with T-Dir Incidence					L-Dir Displ with L-Dir Incidence				
	1.0EQ					1.0EQ				
	B2 $\Delta_{\max}$ (ft)	B3 $\Delta_{\max}$ (ft)	B4 $\Delta_{\max}$ (ft)	B5 $\Delta_{\max}$ (ft)	B6 $\Delta_{\max}$ (ft)	B2 $\Delta_{\max}$ (ft)	B3 $\Delta_{\max}$ (ft)	B4 $\Delta_{\max}$ (ft)	B5 $\Delta_{\max}$ (ft)	B6 $\Delta_{\max}$ (ft)
Yield	2.29	1.33	2.91	2.65	2.13	1.38	1.38	1.49	1.48	1.20
NL Pushover	4.43	4.83	5.33	5.44	5.72	4.07	4.28	4.28	4.61	4.60
ARS	1.97	1.53	1.59	2.09	3.01	1.62	1.62	1.63	1.63	1.63
LTH	1.70	1.65	1.74	2.07	2.95	1.64	1.64	1.64	1.64	1.63
NLTH	1.82	1.78	1.88	2.14	2.65	1.73	1.73	1.73	1.73	1.72
ARS/LTH	1.15	1.08	1.09	0.99	0.98	1.01	1.01	1.01	1.00	1.00
NLTH/LTH	1.07	1.07	1.08	1.03	0.90	1.06	1.06	1.06	1.06	1.05
NLTH/Push	0.41	0.37	0.35	0.39	0.46	0.43	0.40	0.40	0.37	0.37
NLTH/Yield	0.79	1.33	0.65	0.81	1.24	1.26	1.26	1.16	1.17	1.44
ARS/Push	0.44	0.32	0.30	0.38	0.53	0.40	0.38	0.38	0.35	0.35
ARS/Yield	0.86	1.15	0.55	0.79	1.41	1.17	1.18	1.09	1.10	1.36
Column Status	Stable	Stable	Stable	Stable	Stable	Stable	Stable	Stable	Stable	Stable
Substructure of Bridge-4 is column on large diameter CIDH pile shaft. The displacement listed is the absolute displacement including displacement of CIDH pile shaft.										

Figure 7 through Figure 10 showing the linear time history displacement and nonlinear time history displacement of Bridge-1 and Bridge-2 models subjects to ground motion of Imperial Valley EQ, 1979.



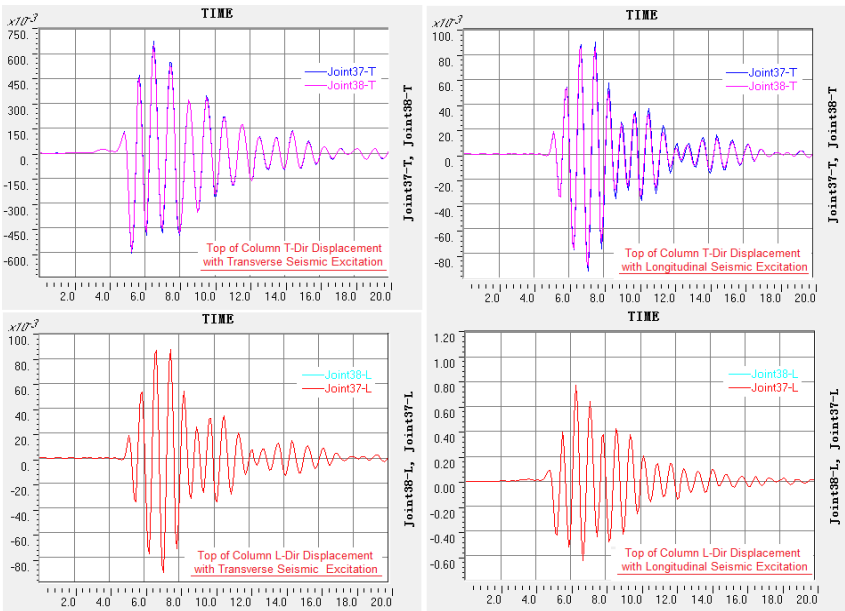


Figure 9. Linear Column Displacement Time History of Bridge-2 (LTH)
Imperial Valley EQ, 1979, Incidence Constant = 1.25

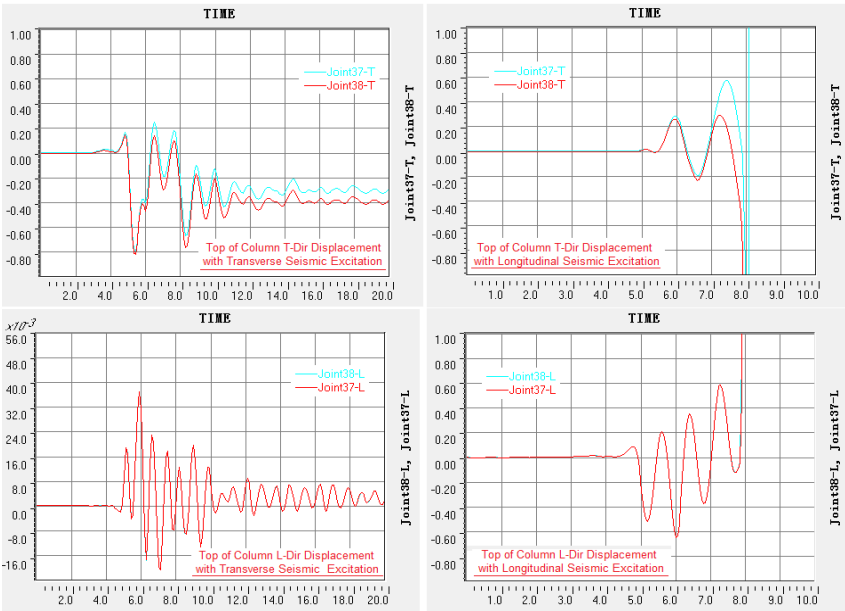


Figure 10. Nonlinear Column Displacement Time History of Bridge-2 (NLTH)
Imperial Valley EQ, 1979, Incidence Constant = 1.25

The discussion of the bridge models analysis results are organized and presented below.

A. Linear Seismic Displacement vs. Nonlinear Time History Displacement

From above tables it can be seen that the ratios of the maximum displacement of ARS analysis to LTH analysis are all about 1.0 as expected. However unlike in the SDF system, the ratios of the maximum displacement of LTH analysis to NLTH analysis are no longer close to 1.0 in most of the analysis cases. By comparing this ratio for Bridge-1 through Bridge-4 in the tables, it is seen that three factors affect this ratio. They are similarity to SDF system, structure regularity, and substructure nonlinearity in seismic response. Bridge-1 is a bridge symmetric in both longitudinal and transverse directions. Its single column bent acts similar to a cantilever beam in transverse direction while multi-bent substructure with superstructure forms a frame in longitudinal direction. Therefore its transverse seismic response is similar to a SDF system and the ratio of NLTH/LTH is seen to be closed to 1.0 while MDF system type response in longitudinal direction has this ratio much smaller than 1.0. Both Bridge-2 and Bridge-3 have torsional dynamic response due to skew and span unbalance irregularity, the ratios of NLTH/LTH have noticeable difference from 1.0. Bridge-4 is a most irregular bridge system among all the analyzed models however its ratios of NLTH/LTH is closed to 1.0 with the difference less than 10% in both longitudinal and transverse direction. The reason is that Bridge-4 substructure columns do not experience much nonlinearity in seismic response to the selected ground motion which can be seen from the ratio ARS/Yield. The ratios of ARS/Yield are all less than 1.5 in table of Bridge-4.

B. Displacement Ductility Ratios ARS/Yield and NLTH/Yield

As discussed earlier in section for SDF system, the ratio ARS/Yield and NLTH/Yield are the indication of the structure demand ductility, it is important in predicting the column nonlinearity in responding to design seismic excitation and the safety. Unlike in SDF system, the ratio of ARS/Yield and NLTH/Yield may have big discrepancy in bridge which has obvious 3-D responding effect, for example, in Bridge-2 and Bridge-3 as shown in Table 7 and Table 8 above. In some cases the ratio NLTH/Yield is larger than ratio ARS/Yield, and reversed in other cases.

Per SDC design criteria with simplified analysis procedure, both Bridge-2 and Bridge-3 are considered as safe in transverse direction when responding to ground motion of Imperial Valley, 1979 with an excitation amplify constant less or equal to 1.5. Because both displacement capacity and ductility capacity is larger than demand, ie. $ARS/Push < 1.0$ and $ARS/Yield < 4.0$. However the NLTH analysis shows that the columns are failed during the seismic event with amplify constant equal to 1.5. In longitudinal EQ response the case with excitation amplifier of 1.25 has the same situation. This observation may alert design engineers that when bridge has high skewed angle or obvious unbalanced span simplify procedure may not be enough for seismic analysis, nonlinear time history analysis may need to be carried out.

C. Linear Seismic Response vs. Nonlinear Seismic Response

In the analysis of this study the bridge deform pattern differences among linear seismic analysis, nonlinear time history analysis and static nonlinear pushover analysis are observed in bridges with noticeable 3-D effect. By comparing the displacement time histories (or step history in static pushover), the cause of the discrepancy seem can be explained.

From Figure 7 and Figure 9, it is seen that the linear transverse displacement time history curves of two column bents have magnitude difference in high peak region and are almost overlap each other in rest of the low magnitude time zone. This means the bridge has torsional 3-D movement when it is in strong seismic response time period, and the response reduces into an in-plan movement in the subsequent weaker response time zone. In nonlinear time history analysis the opposite phenomena is seen. The displacement curve of Joint-37 (Bent-2) and Joint-38 (Bent-3) is obviously separated in lower response magnitude zone from Figure 8 and Figure 10. This implies that the bridge has relatively big torsional movement during nonlinear seismic response. The reason is that when one of the columns is yielded first, it further causes unbalance of the stiffness in substructure system and induces the 3-D or torsional response effect. Obviously this makes the deform pattern different between linear seismic analysis and nonlinear time history analysis. One interesting observation is that even the torsional effect in nonlinear seismic response is large the maximum torsional effect of the linear dynamic analysis is larger than maximum effect in the nonlinear dynamic analysis. The nonlinearity of the system tends to reduce the extreme displacement at corner or end columns. This is so because the corner or end column is softened by the formation of the plastic hinge due to large movement caused by torsion response and this in turn elongates the vibration period of the nonlinear portion of the bridge. The plasticity also limits/reduces the inertial force that transfers through the corner or end columns. The longer structure vibration period also reduces the bridge's response in the subsequent dynamic movement while in the elastic analysis the unbalanced stiffness distribution and value is kept through the whole response history and ends up to have the extreme displacements at corner or end columns. This can be seen from the ratio of Bent-2 displacement to Bent-3 displacement (Δ_2/Δ_3) for Bridge-3. This ratio from nonlinear time history analysis is more close to 1.0 than the ratio from linear dynamic analysis.

From the study it can be concluded that the factors cause the discrepancy between deform pattern of the static nonlinear pushover analysis and linear ARS dynamic analysis is mainly the 3-D effect and the nonlinearity. In ARS analysis there is no stiffness redistribution occurs. In nonlinear static pushover analysis when one or some substructure columns form plastic hinge, the nonlinearity changes the stiffness distribution but the preset lateral force distribution does not change in entire pushover process. Therefore when one or some columns form plastic hinge, the same lateral push force is kept at top of the 'soften' columns and push them even further and the lateral displacement becomes even non-uniformly distributed along the bridge length. In severe case the displacement pattern of the bridge from ARS dynamic analysis can be very different from nonlinear static pushover analysis, as showed in Figure 11. When the bridge is regular all columns form plastic hinge at about the same time and the small discrepancy can be ignored.

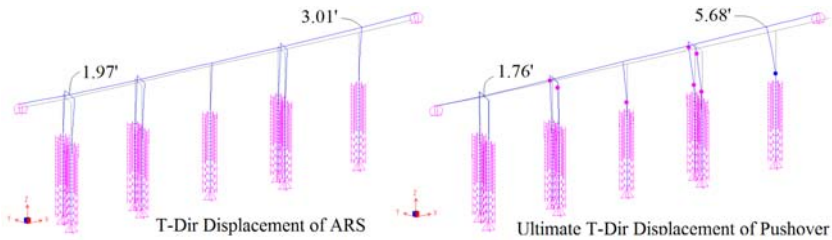


Figure 11. Deform Patterns of ARS Analysis and Nonlinear Static Analysis of Bridge-4

CONCLUSION

Based on above discussions some conclusions can be drawn as following.

1. Popular simplified bridge seismic analysis procedure in current engineering practice works very well in SDF system and it is adequate for design of the OSB category bridge which similar to a SDF system. For these bridges SDC demand ductility limit of 4 for single column bent and 5 for multi-column bent is seen a good threshold.
2. The bridge which possesses MDF system characterizes, elastic linear seismic displacement may not equal or close to its nonlinear dynamic displacement.
3. For bridges in OSB category with high skewed angle or obvious 3-D effect the simplified seismic analysis procedure may not be adequate, and nonlinear time history analysis is strongly recommended. In case the nonlinear time history analysis is not available, reducing the limit of the demand ductility in design may need to be considered.
4. When bridge deform pattern from ARS analysis is very different from its nonlinear static pushover, strong 3-D dynamic effect is implied and nonlinear time history analysis is strongly recommended. In this case design the bridge to have demand ductility close to code limit may overestimate the structural dynamic displacement capacity.
5. Bridge substructure nonlinearity tends to reduce the extreme displacement due to torsional dynamic response at corner or end columns. However it needs more analysis and studies for bridge on the soft soil site with low shear wave velocity.

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Principle of Sliding-Restricting and Its Application in Seismic Design of a Continuous Beam Bridge

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ABSTRACT

To meet the demands of static analysis, fixed bearings that could restrict horizontal displacement were installed in a continuous beam bridge. However, the seismic design of a continuous beam bridge would be rather difficult in high earthquake intensity regions because of fixed bearings. Some measures were taken to improve the seismic performance of continuous beam bridges to some extent, such as replacing all bearings to seismic-isolation bearings. But the seismic safety of continuous beam bridges would not be ensured by these measures when subjected to earthquakes exceeding the preset parameters. Hence, the principle of sliding-restricting was proposed, which was achieved by a Cable-sliding Friction Aseismic Bearing (CFAB). Based on the example of a continuous beam bridge, seismic performance under different types of earthquakes was investigated using a conventional method and a sliding-restricting method. It was found that the initial gap and stiffness of strand cables were two key parameters of CFAB and that the seismic force of a fixed pier could be effectively reduced by CFAB, but seismic displacement at the girder end would be increased.

INTRODUCTION

Continuous beam bridge is one of the most widely-used bridge types in China, and its seismic performance attracts a lot of concern (Li 2008; Huang 2008). According to previous damage investigations, the fixed bearings of continuous beam bridges were damaged because of inadequate shear capacity, which increased the risks of girder falling (Shigeki 2000). However, the seismic forces of fixed piers were increased when the shear capacity of fixed bearings were blindly strengthened. Once the bearing capacity of the fixed piers was exceeded, the fixed piers would be damaged and cause bridges to collapse afterwards. Therefore, rational design of fixed bearings would improve the seismic performance of continuous beam bridges (Bryant 2007).

To meet the demands of static design, only one fixed pier was designed for

continuous beam bridges in the longitudinal direction. During a longitudinal earthquake, a fixed pier would carry almost all the shear force from the superstructure, which made the fixed pier vulnerable to damage. To solve this problem, several methods were provided by scholars (Goutam 2011). Shock Transmission Units (STU) were adopted in sliding piers to disperse the seismic force of a fixed pier (Cassandra 2011), but this method was not suitable for the case when the height of the bridge piers was fairly short. Liquid Viscous Dampers (LVD) were installed at the ends of girders to dissipate the seismic energy in the longitudinal direction (Alexandros 2011), but this method was difficult to popularize because of the high cost of LVD. All conventional bearings were replaced by lead rubber bearings (LRB) to reduce the seismic effects of continuous beam bridges by nonlinear deformation of LRB (Hwang 1996). However, failure of LRB would occur when its allowable displacement was exceeded during an earthquake due to lack of limit function. Similarly, high damping rubber bearings (HDR) and friction pendulum bearings (FPB) were adopted in the seismic design of continuous beam bridges individually to improve the seismic performance of the fixed bearings (Chaudhary 2011; Tsopelas 1996). Like LRB, the risks of bearing failure still remained because HDR and FPB did not have the limit function.

The principle of sliding-restricting was proposed, and Cable-sliding Friction Aseismic Bearing (CFAB) was introduced (Wei 2011). Taking a continuous beam bridge as example, seismic performance was studied that adopted the principles of conventional design and sliding-restriction. Furthermore, a parameter study of CFAB was conducted to obtain a better understanding of the principle of sliding-restriction.

PRINCIPLE OF SLIDING-RESTRICTING

To resist longitudinal loads such as automotive braking force, horizontal bearing capacity was commonly taken as vertical bearing capacity, 10%–20%. However, the shear forces of a fixed bearing might exceed its horizontal bearing capacity during an earthquake, causing damage to the fixed bearing. For pot-type elastomeric pad bearing and spherical bearing, this type of damage was manifested as yielding the displacement limit steel plate of the bearing cover. The movement form of a continuous beam bridge was difficult to predict due to the uncertainty of mechanical behavior of a fixed bearing after destruction. On the other hand, as the horizontal bearing capacity of a fixed bearing increased, although the safety of the fixed bearing could be guaranteed, the seismic forces of a fixed pier would be further increased. Once the pier damage occurred, the risk of bridge collapse would be greatly increased.

The principle of sliding-restricting was proposed to solve the problem of the seismic design of fixed bearings. The principle could be described as follows: Shear failure of a local component of a fixed bearing was allowed to occur, without changing the seismic structural system of a continuous beam bridge. This component did not participate in working after destruction, and the fixed bearing became a sliding bearing. A displacement-restricting device was installed in the fixed bearing to avoid the excessive displacement that occurred after shear failure of the local component of the fixed bearing. A new bearing, named CFAB, was provided according to the principle of sliding-restricting.

CFAB was composed of a shear pin, a strand cable, and a pot-type elastomeric

pad sliding bearing or spherical sliding bearing. The mechanism of CFAB is described as follows (as shown in Figure 1): When the load acting on CFAB was less than the design shear capacity of the shear pin under a serviceability limit state or a minor earthquake, CFAB was employed as a fixed bearing, and the other parts of the bridge maintained elasticity. When the load acting on CFAB was close to the design shear capacity of the shear pin under a moderate earthquake, CFAB was employed as a fixed bearing, and the other parts of bridge should meet the requirements of strength checking. When the load acting on CFAB exceeded the design shear capacity of the shear pin under a rare earthquake, the shear pin was broken, and CFAB was employed as a sliding bearing, which made it easier for the other parts of the bridge to meet the requirements of strength checking; seismic energy could be dissipated by friction caused by sliding of CFAB. When relative displacement between the girder and the pier was equal to the allowable displacement of the strand cable, the strand cable started to work and the relative displacement was restricted.

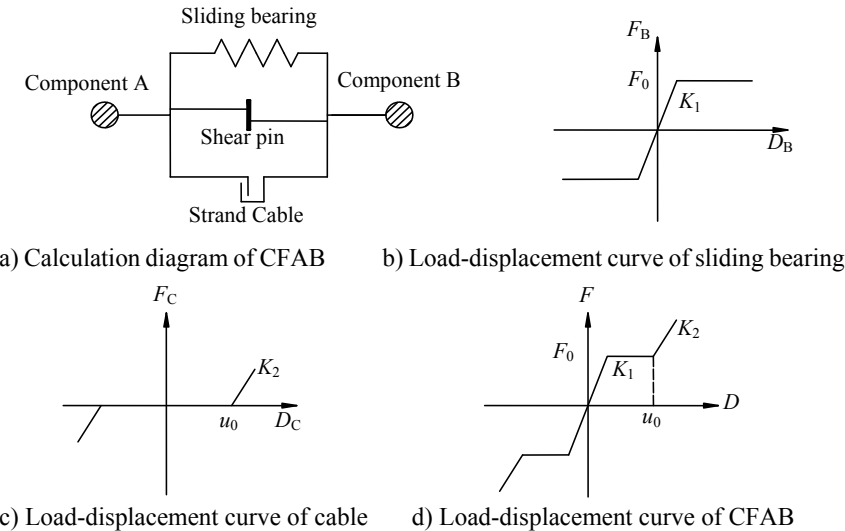


Figure 1. Mechanism of CFAB.

DYNAMIC ANALYSIS MODEL

A continuous beam bridge was chosen as an example, with a span arrangement of $(88+152+88) = 328\text{m}$. A single cell box section was adopted for the girder. The height of the girder was 9.5m at the fulcrum and 4.0m at the middle span, with a gradual change by 2.0 times parabola. The roof width of the girder was 13.65m, and the bottom width was 7.0m. A solid pier was adopted for the main piers, with a thickness of 4.0m. The thickness of the cap below the main pier was 5.0m. A bored pile foundation was employed as the foundation of the main piers, with 6 piles of 2.5m diameter at each pier. Two cylindrical piers were adopted for the transition pier, with a diameter of 2.2m. A bored pile foundation was employed as the foundation of the transition pier,

with a diameter of 2.5m. The arrangement of the bridge bearings was as follows: a bidirectional sliding bearing and a longitudinal sliding bearing were installed in transition piers No. 1 and No. 4 and main pier No. 3; a fixed bearing and transverse sliding bearing were installed in main pier No. 2. The bridge was located in a high intensity seismic zone. Figure 2 shows a horizontal earthquake acceleration time-history curve, with a return period of 1950 and a damping ratio of 0.05.

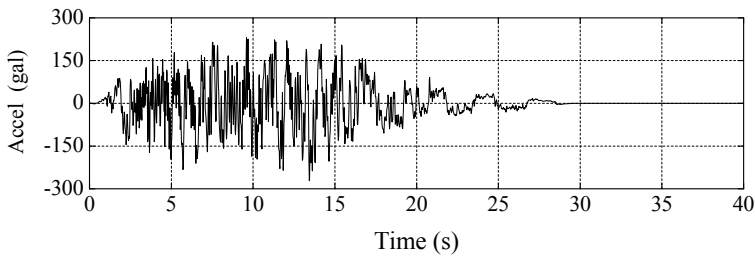


Figure 2. Seismic input.

A dynamic analysis model of the continuous beam bridge is shown in Figure 3. A girder, cap beam, pier, tie beam, and cap and pile foundation were simulated by a space beam element. Pile soil interaction was considered by spring with “m-method.” Boundary conditions were simulated by actual location and performance of bearings. A fixed connection was used between the girder and the top of main pier No. 2 in the longitudinal direction. A sliding connection was used between the girder and the top of other piers in the longitudinal direction. A fixed connection was used between the girder and the top of all piers in the transverse and vertical directions.

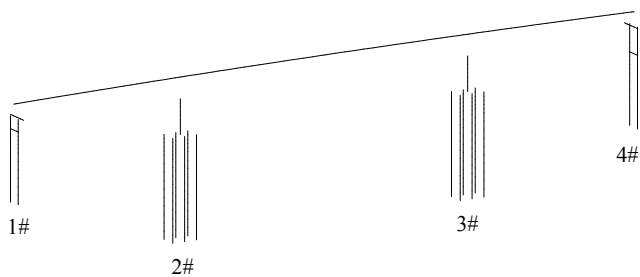


Figure 3. Dynamic analysis model.

SEISMIC PERFORMANCE ANALYSIS

Based on dynamic analysis model and seismic input, a time-history analysis method was adopted to investigate the seismic performance of the bridge by using a conventional method and a sliding-restricting method individually. When the sliding-restricting method was employed, two bearings installed in main pier No. 2 were replaced by CFAB. The parameters of CFAB were as follows: the yielding force of shear pin was 9000kN; the yielding force of sliding bearing F_0 was 1350kN; the

friction stiffness K_1 was 675000kN/m (yielding displacement was 0.002m); the initial gap u_0 was 0.05m; the stiffness of strand cable K_2 was 200000kN/m.

The calculated longitudinal seismic displacement at the girder end, using the conventional method and the sliding-restricting method, is shown in Figure 4. The maximum longitudinal seismic displacement using the conventional method was 148.6mm and was 178.3mm using the sliding-restricting method. It can be seen that the longitudinal seismic displacements at the girder end under the two different methods were not large, within the limits of ± 200 mm; compared with the conventional method, the maximum longitudinal seismic displacement could be increased by the sliding-restricting method, with an increase of 20.0%; however, the vibration frequency of the bridge could be reduced by the sliding-restricting method when compared with the conventional method.

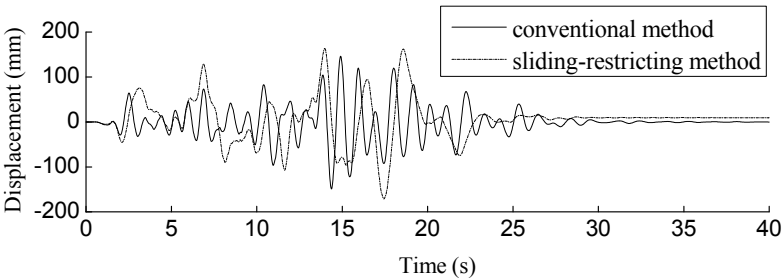


Figure 4. Time-history curve of displacement at girder end.

The calculated seismic moment at the bottom of the main pier No. 2 using the conventional method and the sliding-restricting method is shown in Figure 5. The maximum longitudinal seismic moment using the conventional method was 924.6 MN • m and 369.1 MN • m using the sliding-restricting method. It could be seen that, compared with the conventional method, the maximum longitudinal seismic moment could be dramatically decreased by the sliding-restricting method, with a decrease of 60.1%; when subjected to vibration frequency, the same rule was shown between the longitudinal seismic moment and the longitudinal seismic displacement. The reason is that the seismic structural system was changed by the sliding-restricting method, from a fixed system to a sliding system, which could dissipate earthquake energy and reduce the seismic effects on the bridge.

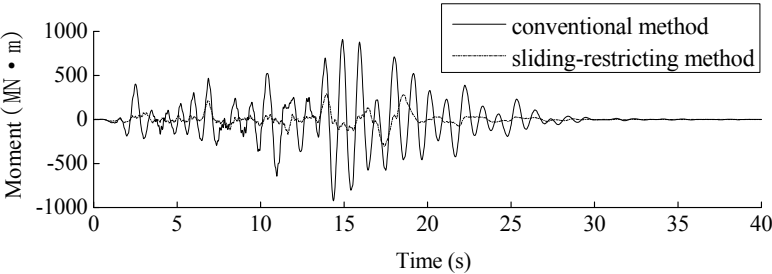


Figure 5. Time-history curve of moment at bottom of main pier No. 2.

PARAMETER ANALYSIS

Parameter analysis was conducted for CFAB to investigate its influence on the seismic performance of a continuous beam bridge. The parameters of CFAB were a yielding force of sliding bearing F_0 , a friction stiffness K_1 , an initial gap u_0 , and a stiffness of strand cable K_2 . Among them, F_0 could be chosen by bearing type, and K_1 could be calculated by choosing a yielding displacement as 0.002m. Hence, the main parameters of CFAB were u_0 and K_2 , values of which are shown in Table 1.

Table 1. Parameter values of CFAB.

Parameter name	Value
u_0 (m)	0, 0.05, 0.1, 0.15, 0.2
K_2 (kN/m)	5×10^4 , 10×10^4 , 15×10^4 , 20×10^4

The impact of the parameters on displacement at the girder end is shown in Figure 6. It can be seen that, as u_0 increased, maximum displacement gradually increased. When u_0 increased to a certain extent, increasing the rate of maximum displacement slowed down. The reason is that the constraint between CFAB and the girder became weaker as u_0 increased. When the effects of constraint were almost no effect, increasing the rate of maximum displacement was close to zero. As K_2 increased, maximum displacement gradually decreased. When K_2 increased to a certain extent, decreasing the rate of maximum displacement slowed down. The reason is that constraint of CFAB to the girder became stronger as u_0 increased. When the effects of constraint were strong enough to some extent, decreasing the rate of maximum displacement was close to zero.

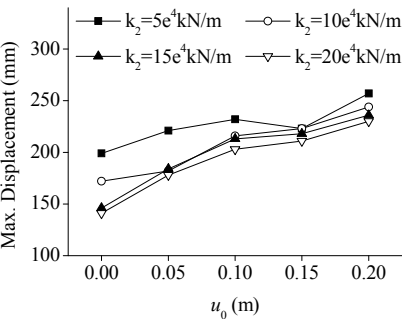


Figure 6. Impact of parameters on displacement at girder end.

The impact of parameters on moment at the end of main pier No.2 is shown in Figure 7. It can be seen that, as u_0 increased, maximum moment gradually decreased. When u_0 increased to a certain extent, decreasing the rate of maximum moment slowed down. The reason is that constraint between CFAB and the girder became weaker as u_0 increased. When the effects of constraint were almost no effect, increasing the rate of maximum moment was close to zero. As K_2 increased, the maximum moment gradually increased. When K_2 increased to a certain extent, increasing the rate of

maximum moment slowed down. The reason is that constraint of CFAB to the girder became stronger as u_0 increased. When the effects of constraint were strong enough to some extent, decreasing the rate of maximum moment was close to zero.

Finally, the parameters of CFAB were chosen as $u_0=0.15\text{m}$, $K_2=15\times 10^4\text{kN/m}$.

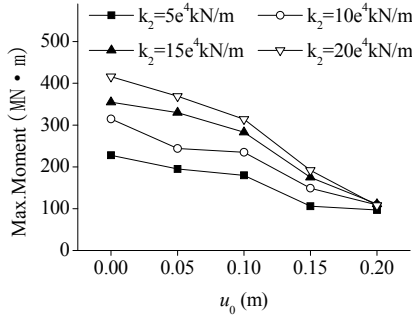


Figure 7. Impact of parameters on moment at pier bottom.

CONCLUSIONS

1) The principle of sliding-restricting allowed the fixed bearing to slide after shear failure, but it did not allow a too-large displacement. The seismic force of a fixed pier could be effectively reduced by this principle, but the seismic displacement at the pier end would be increased.

2) CFAB, designed by the principle of sliding-restricting, could control the required force of shear failure by a shear pin and limit the too-large seismic displacement by a strand cable.

3) The initial gap and stiffness of a strand cable are the main parameters of CFAB. Seismic displacement at the pier end increased as the initial gap increased, and decreased as stiffness of the strand cable increased. The moment at the bottom of the fixed pier decreased as the initial gap increased, and increased as stiffness of the strand cable increased. The influence of CFAB on the seismic performance of a continuous beam bridge was close to zero when the initial gap or stiffness of the strand cable increased to a certain extent.

ACKNOWLEDGMENTS

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Behavior of Multi-Spherical Sliding Friction Isolation Bearings

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ABSTRACT

A novel multi-spherical sliding friction isolation bearing for seismic design of bridges is developed in this paper. The configuration characteristic, operation and sliding mechanism are explained. The nonlinear restoring model is presented and the simplified calculation formula of force-displacement relationship of the bearing with identical friction coefficient of frictional interface is established. Experiment of mechanics performance is carried out and the sliding order and the force-displacement relationship are verified. The results show sliding order in theoretical analysis and experimental observation is identical and force-displacement relationship curve derived from theoretical analysis is well agreed with experimental results. The multi-spherical sliding friction isolation bearing has good energy dissipation capacity, horizontal displacement capacity, designated re-centering function, smart stiffness and smart damping behavior under different intensity amplitudes external excitation as it can optimize the parameters of an isolation system for a given bridge based on multiple performance objectives or multiple earthquake levels. These characteristics make the proposed bearing suits for performance-based seismic design of bridges for multiple levels of ground shaking.

INTRODUCTION

In bridge seismic isolation technology, the application of isolation bearings instead of ordinary bearings is a both economic and effective way in bridge engineering seismic design. The current isolation bearing mainly includes three types, including: laminated rubber bearings, viscosity bearings and sliding friction isolation bearings. As the sliding friction isolation bearing has great tonnage load capacity, large displacement and good durability, especially can play a role in avoiding resonance phenomenon of the bridge structure, it attracts the bridge engineer's and researcher's attention in recent years.

Some research is available in the literature on the sliding friction isolation bearing. For three kinds of this bearing, involving (1) Friction pendulum bearing (Zayas et al. 1990; Mokha et al. 1990; Mokha et al. 1991; Tsopelas et al. 1996). (2)

Double concave friction pendulum (Tsai et al. 2003; Tsai et al. 2005; Constantinou et al. 2004; Constantinou et al. 2005; Kim et al. 2007). (3) Triple friction pendulum bearing (Fenz et al. 2008; Fadi et al. 2010; Deng Xue-song et al. 2010; Deng Xue-song et al. 2011). Its mechanics performance, restoring force model and the seismic response of bridge applied such isolation bearing have been studied both experimentally and theoretically.

Based on the principle of multi-friction isolation bearing, a novel multi-spherical sliding friction isolation bearing is developed in this paper. The configurational characteristic, operational and sliding mechanisms of this bridge isolator are explained. The nonlinear restoring model and the simplified calculation formula of force-displacement relationship with identical friction coefficient of frictional interface of the bearing are presented. The experiment of mechanics performance of this isolator is carried out, and sliding order of all frictional face and force-displacement relationship of this bearing is verified. Stainless steel, as the main material of the bearing, promotes load capacity and durability. The friction pendulum model of the bearing has certain re-centering function. Moreover, the stiffness and the damping which related to the two parameters spherical curvature radius and friction coefficient are smart, as they can be changed by self-adaptability. It is expected to be applied for multi-level seismic isolated bridge under performance-based seismic design.

1. CONFIGURATIONAL CHARACTERISTIC AND WORKING MECHANISM OF ISOLATION BEARING

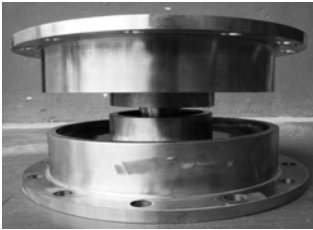
The multi-spherical sliding friction isolation bearing as shown in fig.1 consists of upper and lower bearing plates, nested upper and lower sliders and a cylindrical slider. Each bearing plate has a spherical concave which is defined as surface 1 and 4 respectively. They have the same radius of curvature, namely $R_1 = R_4$. And each nested slider also has a spherical concave which defined as surface 2 and 3 respectively. They also have the same radius of curvature, namely $R_2 = R_3$. Then R_i is the curvature radius of the i th spherical surface, and μ_i is the friction coefficient of each surface. The outer surfaces of both nested sliders and rigid cylindrical slider are coated with a non-metallic material called polytetrafluoroethylene (PTFE) as to control the friction factor. There are two retainer rings on the upper and lower concave plates as displacement restrainer. Sliding will be stopped by the retainer ring when impact occurs. The maximum displacement D_i (also called nominal displacement capacity) of every surface is

denoted as $D_1 \sim D_4$. Therefore, the maximum displacement of the bearing is $\sum_{i=1}^4 D_i$.

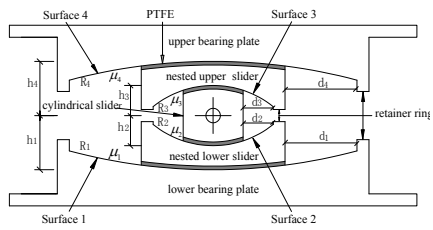
Heights, h_i , is the radial distance between the i th spherical surface and the pivot point of the articulated slider. Here, the effective curvature radius of i th surface is $R_{ei} = R_i - h_i$, accounting for the factor of h_i .

The working mechanism of this bearing can be organized into several sliding regimes, each corresponding to a distinct sliding combination. Firstly, Sliding does not initiate on the i th surface until the lateral force F exceeds its maximum static friction

force. Secondly, according to the literature (Zayas et al. 1990 and Tsopelas et al. 1996), the stiffness of the bearing is inversely proportional to the curvature radius for friction pendulum bearing. That is $K = P/R_e$ (in this paper, the formula derivation assumes that $\sin \alpha = \tan \alpha = \alpha$, $\cos \alpha = 1$ since the rotation angle α is small enough), where P is the vertical force and R_e is the effective curvature radius. Similarly, the stiffness of the multi-spherical sliding friction isolation bearing is $K = P/(R_{ei} + R_{ej})$. Thirdly, we have to emphasize that the relationship between friction coefficient and curvature radius must be well designed in order to maintain the re-centering function of the bearing. The maximum residual displacement is $X_{\max - \text{remain}} = \sum_{i=1}^4 \mu_i R_i$.



(a) The photo of isolation bearing



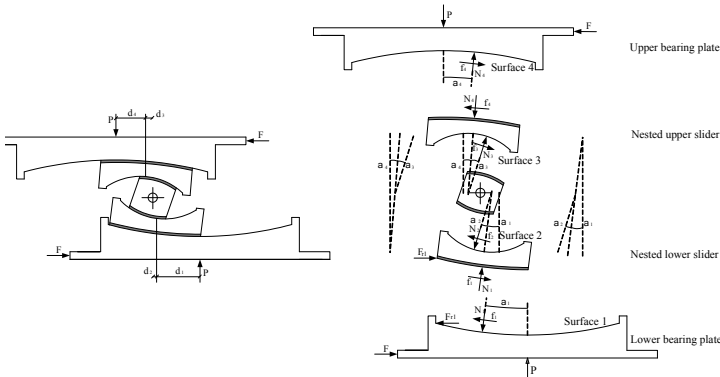
(b) The cross-sectional view

Figure 1. The Multi-Spherical Sliding Friction Isolation Bearing

2. FORCE-DISPLACEMENT RELATIONSHIP OF ISOLATION BEARING

2.1 The bearing with different spherical friction coefficient

Given that: $\mu_4 \geq \mu_1 \geq \mu_2 = \mu_3$. The sliding regimes and the force-displacement relationship can be revealed clearly according to the force equilibrium relationship and the dynamic geometric relationship. Figure2 shows the cross section and sliding state diagrams of isolation bearing.



(a) Displacement and force of the bearing

(b) The sliding state diagrams

Figure 2. Cross Section and Sliding State Diagrams of Isolation Bearing

Sliding regime 1: When $F = f_2 = f_3$, sliding initiates only on surface 2 and 3, while surface 1 and 4 keep static. The displacement is a combination of surface 2 and 3 sliding. The vertical and horizontal force equilibrium equations of the nested lower sliding plate are:

$$\begin{cases} N_1 + f_2 \sin \alpha_2 - N_2 \cos \alpha_2 = 0 \\ f_2 \cos \alpha_2 + N_2 \sin \alpha_2 - f_1 = 0 \end{cases} \xrightarrow{\alpha \text{ 很小}} \begin{cases} F = f_1 \\ P = N_1 \end{cases} \quad (1)$$

Where N_i is the relative vertical force to i th sliding surface. α_i is the relative angle.

For the subject of lower bearing plate and nested lower slide, and the lower bearing plates, their horizontal force equilibrium equations are:

$$\begin{cases} F = (P/R_{e2})d_2 + f_2 \\ F = (P/R_{e3})d_3 + f_3 \end{cases} \rightarrow \begin{cases} d_2 = (F - f_2)R_{e2}/P \\ d_3 = (F - f_3)R_{e3}/P \end{cases} \quad (2)$$

The total displacement of the bearing is the sum of d_2 and d_3 , then we can get the force-displacement relationship of sliding regime 1:

$$F = \frac{P}{R_{e2} + R_{e3}}d + \frac{f_2 R_{e2} + f_3 R_{e3}}{R_{e2} + R_{e3}} \quad (3)$$

Sliding regime 2: When the horizontal force increases to be equal to the friction force of sliding surface 1, that is $F = f_1$, sliding commences on surface 1 and 3, while surface 2 and 4 keep static. The displacement of the bearing at the finish of sliding regime 1 is given:

$$d_{12} = (\mu_1 - \mu_2)R_{e2} + (\mu_1 - \mu_3)R_{e3} \quad (4)$$

Where d_{i+1} means the sliding state changes from i ($i = 1, 2, 3, 4$) to $i + 1$.

As for lower bearing plate and lower sliding plate, the horizontal force equilibrium formulations are:

$$\begin{cases} F = (P/R_{e1})d_1 + f_1 \\ F = P[(d_1/R_{e1}) + (d_2/R_{e2})] + f_2 \end{cases} \rightarrow \begin{cases} d_1 = (F - f_1)R_{e1}/P \\ d_2 = (\mu_1 - \mu_2)R_{e2} \end{cases} \quad (5)$$

Equation $\overline{d_2} = (\mu_1 - \mu_2)R_{e2}$ also shows that sliding surface 2 stops once the surface 1 begins to slide.

The sliding displacement on surface 2 is no longer the d_1 but the $\overline{d_2}$, then the total displacement of the bearing is $d = d_1 + \overline{d_2} + d_3$, so the force-displacement relationship of sliding regime 2 can be obtained:

$$F = \frac{P}{R_{e1} + R_{e3}}d + \frac{f_1(R_{e1} - R_{e2}) + f_2 R_{e2} + f_3 R_{e3}}{R_{e1} + R_{e3}} \quad (6)$$

Sliding regime 3: when $F = f_4$, sliding commences on surface 4, while sliding on surface 3 stops. The displacement is a combination of surface 1 and 4 sliding. Substituting $F = f_4$ into the formula (6), we can get:

$$d_{23} = d_{12} + (\mu_4 - \mu_1)(R_{e1} + R_{e3}) \quad (7)$$

Similar to the analysis above:

$$\begin{cases} F = (P/R_{e4})d_4 + f_4 \\ F = P[(d_3/R_{e3}) + (d_4/R_{e4})] + f_3 \end{cases} \rightarrow \begin{cases} d_4 = (F - f_4)R_{e4}/P \\ \overline{d_3} = (\mu_4 - \mu_3)R_{e3} \end{cases} \quad (8)$$

Substituting d_i ($i = 1, 2, 3, 4$), into $d = d_1 + \overline{d_2} + \overline{d_3} + d_4$, the force-displacement

relationship of sliding regime 3 is:

$$F = \frac{P}{R_{e1} + R_{e4}} d + \frac{f_1(R_{e1} - R_{e2}) + f_2 R_{e2} + f_3 R_{e3} + f_4(R_{e4} - R_{e3})}{R_{e1} + R_{e4}} \quad (9)$$

Sliding regime 4: When $d_1 = D_1$, the nested lower slider impact the retainer ring of the lower bearing plate, the sliding on surface 1 stops and sliding on surface 2 restarts. The total displacement is a combination of surface 2 and 4 sliding. The nominal displacement of surface 1 reaches its maximum value D_1 , while the horizontal force on surface 1 is:

$$F_{s1} = (P/R_{e1}) D_1 + f_1 \quad (10)$$

When $F = F_{s1}$, the displacement of the bearing at the finish of sliding regime 3 is given:

$$d_{34} = d_{23} + D_1(1 + \frac{R_{e4}}{R_{e1}}) - (\mu_4 - \mu_1)(R_{e1} + R_{e4}) \quad (11)$$

Similar to the analysis above:

$$\begin{cases} F = (P/R_{e1}) D_1 + f_1 + F_{r1} \\ F = P[(D_1/R_{e1}) + (d_2/R_{e2})] + f_2 \end{cases} \rightarrow \overline{d_2} = (F_{r1} + f_1 - f_2) R_{e2} / P \quad (12)$$

Substituting the formula $F = (P/R_{e1}) D_1 + f_1 + F_{r1}$ (17) into $d_4 = (F - f_4) R_{e4} / P$ (8) and get $d'_4 = (R_4/R_1) D_1 + \mu_1 R_4 - \mu_4 R_4 + (F_{r1}/P) R_4$. The total displacement is $d = D_1 + \overline{d_2} + \overline{d_3} + d'_4$, that is:

$$d = \frac{F_{r1}(R_{e2} + R_{e4})}{P} + D_1 + \mu_1 R_{e2} - \mu_2 R_{e2} + \mu_4 R_{e3} - \mu_3 R_{e3} + \frac{R_{e4}}{R_{e1}} D_1 + \mu_1 R_{e4} - \mu_4 R_{e4} \quad (13)$$

Then, the force-displacement relationship of sliding regime 4 is:

$$F = \frac{P}{R_{e2} + R_{e4}} (d - d_{34}) + \frac{P}{R_{e1}} D_1 + f_1 \quad (14)$$

Sliding regime 5: When $d_4 = D_4$, the sliding on surface 4 stops and the sliding on surface 3 restarts once the nested upper plate impact the retainer ring of the upper bearing plate. The total displacement is a combination of surface 2 and 3 sliding. The nominal displacement of surface 1 and 4 reaches its maximum value D_1 and D_4 . Similarly, we can get:

$$d_{45} = d_{34} + [(\frac{D_4}{R_{e4}} + \mu_4) - (\frac{D_1}{R_{e1}} + \mu_1)](R_{e2} + R_{e3}) \quad (15)$$

$$F = \frac{P}{R_{e2} + R_{e3}} (d - d_{45}) + \frac{P}{R_{e4}} D_4 + f_4 \quad (16)$$

The whole force-displacement of the bearing including each sliding state, in case of the relationship of friction coefficients is $\mu_4 \geq \mu_1 \geq \mu_2 = \mu_3$, is shown in Figure3 (a).

2.2 The bearing with the same spherical friction coefficient

If $\mu_1 = \mu_2 = \mu_3 = \mu_4$, the order of the slide changes. Sliding initiates from all sides at the very beginning. However, the sliding on surface 1 and 4 is the main factor for consideration and the sliding on surface 2 and 3 can be neglected after sliding occurs, since the curvature radius differ greatly which causes a large stiffness difference, it makes the sliding on the surfaces 2 and 3 much smaller than the sliding on surface 1 and

4. The sliding on surface 2 and 3 doesn't become the main factor until the impact occurs. The force-displacement relationship is given:

When $d \leq D_1 + D_4$,

$$F = \frac{P}{R_{e1} + R_{e4}} d + f \tag{17}$$

When $D_1 + D_4 < d \leq D_1 + d_2 + d_3 + D_4$,

$$F = \frac{P}{R_{e2} + R_{e3}} d - \frac{P(D_1 + D_4)(R_{e1} + R_{e4} - R_{e2} - R_{e3})}{(R_{e1} + R_{e4})(R_{e2} + R_{e3})} + f \tag{18}$$

The whole force-displacement of the bearing in case of all the friction coefficients are equal is shown in Figure3 (b).

Figure 3 (a) shows that there are four stiffness changes for the bearing with different friction coefficients. There are little changes on previous stiffness. The original polyline will become closer to straight line or curve nearly straight [13] and the stiffness change will not be as clear as of the theoretical analyze. Moreover, the hysteresis curve inflection point is not clear with the increase of loading. These make it difficult for application. In contrast, Figure3 (b) reveals a conventional bilinear model of force-displacement relationship when $\mu_1 = \mu_2 = \mu_3 = \mu_4$. Its second stiffness is greater than the first stiffness, namely conventional bilinear stiffness hardening model. That because the stiffness is mainly determined by the effective radius of curvature rather than friction coefficients. Bridge engineering technical personnel are very familiar with the bilinear hardening model for use.

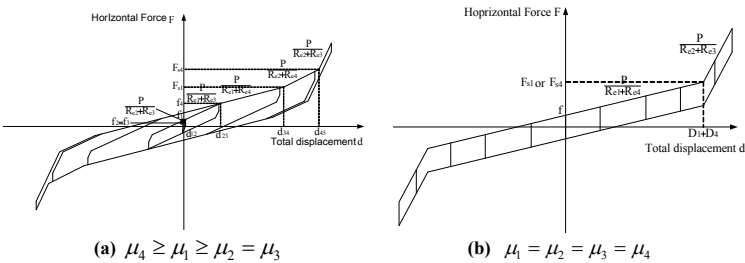
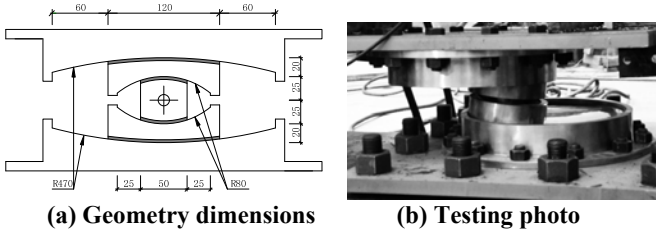


Figure 3. Load-Displacement Hysteretic Curves of Isolation Bearing

3. EXPERIMENTAL CONFIRMATION

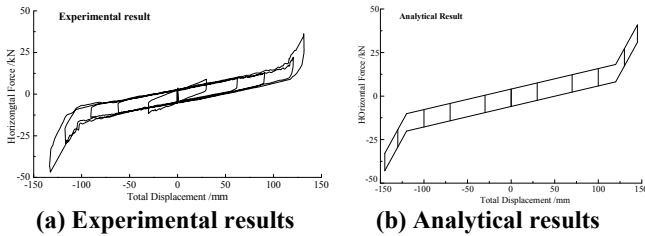
The experiments presented in this paper were tested at the Key Laboratory of Urban Security and Disaster Engineering of Ministry of Education, Beijing University of Technology. Combined with the actual situation of the specific projects, the friction coefficient of the sliding bearing surfaces of the specimen is 0.04 and the test pressure at a constant vertical bearing is 100kN, with a loading speed 1mm / s. The geometry dimensions and testing photo of isolation bearing are shown in Figure4.



(a) Geometry dimensions **(b) Testing photo**
Figure 4. Geometry Dimensions and Testing Photo of Isolation Bearing (Unit: mm)

The results shows sliding order of all frictional face of bearing in theoretical analysis and experimental observation is identical and force-displacement relationship curve of bearings derived from theoretical analysis is well agree with experimental results. The motion state as show in Figure2 corresponds to the state of sliding mode as shown in Figure4. Experimental and analytical load-displacement hysteretic curves of specimen bearing are very consistent, as shown in Figure5.

Meanwhile, the hysteresis curve is quite full and the shape is rather regular, indicating a good energy dissipation capacity, having sufficient horizontal displacement, as well. For medium to small degree of external excitation, seismic isolation mechanism mainly relies on the sliding surface 1 and 4, in which case there is less stiffness and large displacement space. But for large degree of external excitation, it mainly relies on the sliding surface 2 and 3, exhibits greater stiffness. Therefore, it may exhibit different excitation stiffness characteristics depending on the size of the external support, which means a good self-adaptive performance of the bearing.



(a) Experimental results **(b) Analytical results**
Figure 5. Experimental and Analytical Load-Displacement Hysteretic Curves

4. CONCLUSIONS

(1) A novel multi-spherical sliding friction isolation bearing is developed in this paper. It has good energy dissipation capacity, horizontal displacement capacity, designated re-centering function, smart stiffness and smart damping behavior. The bearing suits for performance-based seismic design of bridges for multiple levels of ground shaking.

(2) This paper theoretically establishes its nonlinear restoring force model and the simplified formula of the force - displacement relationship. Experiment of mechanics performance is carried out and the sliding order and the force-displacement relationship are verified in case of all the friction coefficients are equal.

(3) The hysteresis curve of the multi-spherical friction isolation bearing in case of all the friction coefficients are equal is quite full and the shape is rather regular, indicating a good energy dissipation capacity, the sufficient horizontal displacement. It may exhibit different excitation stiffness characteristics, depending on the intensity amplitudes external excitation, which means smart stiffness and smart damping behavior. Furthermore, the simplified bilinear stiffness hardening model is practical.

(4) More theoretical analysis and experimental research need to be carried out on some important factors such as its dynamic performance, the parameter of friction coefficient and the radius of curvature, the space between nested slider and retaining ring etc.

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A State-Of-The-Art Review of Seismic Isolation Technology in Bridge Engineering

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ABSTRACT

Seismic isolation technology has been developed for several decades from the 1970s. Massive research works on seismic isolation devices, isolation theory and engineering practice were performed by scholars and engineers, and abundant achievements were acquired in this field. Development of bridge isolation technology is briefly summarized in this paper firstly, and several isolation devices for bridge engineering are introduced including some novel isolators developed in recent years. Isolation fundamental theory researches in bridge engineering on near-fault effect, collision effect, experiment technology and new design methods are reviewed in this paper to provide references for next generation research on isolation technology in bridge engineering. At the end, recommendations and discussion are proposed for future research and practice.

INTRODUCTION

Bridge structures are pivotal to transportation thus they are among the most important research contents in urban disaster prevention and mitigation. Taking reasonable and effective seismic measures to ensure the safety and normal use of bridge structures during earthquakes are significant to the urban anti-earthquake and post-earthquake reconstruction. The substructure and the bearing are most likely to be damaged. Collapse of whole bridge caused by destruction of pier and unseating of superstructure caused by insufficient deformation capacity of bearing and other destruction of bridge structure often occurs in earthquake. Currently, the seismic design of bridges mainly aims to dissipate the seismic energy by its own elements (plastic hinge of pier) or damping devices (e.g. damper) and to reduce the responses of bridges by adopting isolation bearings (e.g. lead rubber bearing), which can avoid dominant frequency of seismic excitations by prolonging the nature vibration period of bridge structures. Generally, adopting the seismic isolation technology is a good alternative method, bridge structure can use isolation bearings to replace the common bridge bearing. The isolation bearing can

meet the performance requirements of bearing capacity, environmental loads (e.g. wind load) and minor earthquake to protect bridge structures from yielding or harmful vibration. Also during earthquake impact, the isolation bearing can dissipate some seismic energy and prolong the nature vibration period, thus can reduce the inertia force and protect the main structure from serious damage. Additionally, according to the demand of seismic performance, bridge structures can be reinforced by simply replacing appropriate bearing or some other methods. Therefore, adopting the seismic isolation technology is an economic and effective way for bridge seismic design.

DEVELOPMENT OF ISOLATION BEARINGS FOR BRIDGE STRUCTURES

The Laminated Natural Rubber Bearing (LNRB) and The Lead Rubber Bearing (LRB) are most common used isolation bearing in isolated bridge practice during past several decades. LNRB was proposed by an American scholar Kelly in the 1960s. The two isolation bearings are all made by combining layers of rubber and sheet steel alternatively together by HTV technology, in which the Natural Rubber or Neoprene is generally adopted, but LRB isolator is made by inserting one or more lead cores in the center of LNRB, proposed by a New Zealand professor Robinson in 1975 (Robinson and Tucker, 1977). The two isolators all have a large bearing capacity and stiffness in vertical direction, also it has good flexibility in horizontal direction, be able to withstand large shear deformation without losing its stability and keep a small horizontal stiffness. The lead core in LRB can provide the bearing with not only large initial stiffness but also large hysteretic energy dissipation, reducing the relative shear deformation of it. The equivalent damping ratio can reach about 20%-30%.

The High Damping Rubber Bearing (HDRB) uses high damping rubber, which is made by mixing graphite into natural rubber. By controlling the proportion of the graphite, the damping behavior can be regulated. The HDRB has favorable horizontal shear stiffness and damping property, the damping ratio can reach 10%-15%. Currently, the research of the HDRB mainly focuses on the hysteresis mechanical performance test (Zhuang et al., 2006), theoretical model and method of determining parameters, etc. The Yama-age Bridge built in Tochigi-prefecture, Japan in 1992 adopted the HDRB and ten years later, two HDRB were replaced by new bearings. and tested on their shear performance and hysteretic behavior. Comparing the results with that ten years ago, it was discovered that the horizontal shear stiffness was increased by 3%-4%, minor changes in equivalent damping ratio and vertical stiffness, and the ultimate shear performance still met the demand of the displacement requirement (Nishi, 2004).

The Sliding Friction Isolation Bearing (SFIB) is an effective isolation devices, seismic energy can be dissipated by the friction between sliding surfaces. The horizontal shear stiffness of SFIB will decrease significantly once shear force exceeds its initial friction force. The SFIS is not sensitive to the input seismic

excitation frequency, thus maximum horizontal shear force of substructure is approximately equivalent to the sliding friction force, accompanying a notable horizontal shear displacement. If the sliding friction surface is plain and the elastic restoring force is absent, the sliding friction isolation bearing cannot restore its original position. To make up this defect, the Elastic Sliding Isolation Bearing (Lin and Tadjbakhsh, 1990) and the Friction Pendulum Isolation System (FPS) (Jiao et al., 2007) and some other bearings were proposed. Structures with large height-width ratio, bridges with large distance between the superstructure barycenter and bearing may result in a tensile behavior of the isolation bearing. Thus the anti-uplift FPS has been developed in recent years (Rossis and Constantinou, 2005). Then, some scholars improved the existing FPS and proposed the variable curvature friction pendulum isolation system (Tsai et al., 2006), the double concave friction pendulum isolation system (Malekzadeh and Taghikhany, 2010; Deng et al., 2010) and the triple curvature friction pendulum isolation system. Also some research on the working mechanism, dynamic characteristics and engineering application of these proposed systems were performed.

THEORETICAL RESEARCHES OF BRIDGE ISOLATION

The implementation feasibility and effectiveness analysis of intelligent seismic isolation technology is an important field of current research. The swing mechanism isolation system is made by placing a swing system between the pile cap and the end of the pier. When the overturning moment caused by earthquake become larger than the restoring moment provided by self-weight, the pier begins to swing to reduce the vibration energy. Theoretical analyses and model tests show that the swing system can effectively and stably protect high piers in a strong earthquake. Thus the swing system is among those technologies with favorable development prospects (Chen, 2006). The MR viscous fluid damper can be used with the LNRB or the sliding friction bearing together to form a combined isolation system. The theoretical analyses and model testing show that the MR damper and other intelligent dampers can effectively restrain the displacement response of bridge structure, especially for bridges in near-fault regions to control the large displacement response without increasing the shear force. The combined isolation system has a better control effect on seismic isolation bridge structures (Lee and Kawashima, 2004; Sahasrabudhe and Nagarajaiah, 2005). The Shape Memory Alloy (SMA) has a character with super elasticity and impressive energy dissipation capacity. In recent years the SMA has attract comprehensive attention in building and bridge vibration control field. Ozbulut and Hurlbeaus (2011a and 2011b) have analyzed the parameter sensitivity and effectiveness about the combined isolation system made of SMA with LNRB or flat steel-PTFE sliding friction bearing in bridge structures.

The near-fault pulse-type effects with long period is likely to have a distinct effect on isolated bridges. In the 1999 Duzce earthquake in Turkey, the

2.3km-long Bolu viaduct with seismic isolation technology suffered from the near-fault seismic pulse-type excitations, and the surface rupture caused by the earthquake occurred right across the viaduct. As a result, the bridge was significantly affected by the directivity and fling-step effect, leading to a failure of the isolation system. The horizontal displacement of superstructure, with a large permanent horizontal displacement, was greater than the designed maximum displacement. Later on, the assessment of its isolation system showed that it cannot meet with the demand of the seismic isolation specification in AASHTO; and even designed as the AASHTO demands, the viaduct would still be severely damaged due to the near-fault seismic excitations (Roussis et al., 2003). This shows that the near-fault ground motion has a significant effect on isolated bridges in near-fault regions. Professor Liao, a professor in the Graduate Institute of Civil and Disaster Prevention Engineering in NTUT, investigated and got a conclusion that the near-fault ground motion remarkably increases the demanded ductility of bridge pier, and the PGV/PGA ratio can evidently affect the seismic responses of isolated bridges, causing great displacement responses (Liao et al., 2004). Wesolowsky and Wilson (2003) shows that the seismic response of near-fault bridge structures differs from the seismic excitations, the coefficient of variation may reaches 50%. Isolation design of cable-stayed bridges should carefully consider the near-fault ground motion. Jangid (2005 and 2007) investigated seismic responses of near-fault multi-story buildings and bridges with FPS and LRB. The results showed that for an FPS with small friction coefficient and LRB with 1 yield force, the superstructure has a remarkable displacement under near-fault excitations. To reach the minimum acceleration and displacement, the FPS has an optimal friction coefficient and the LRB has an optimal yield force. Dicleli and Buddaram (2007) investigated the effect of isolator properties, the frequency character and intensity of ground motions on isolated bridge responses. The results showed that the seismic response of isolated bridge structures is a function of PGV/PGA, the seismic design should base on the site of the bridge, the design of the characteristic strength of seismic isolators should base on the frequency character and intensity of ground motions. Also, the post-yield stiffness of the isolator has distinct effect on seismic responses of bridges. To reduce the displacement of isolators and meanwhile keep the seismic force of superstructure in a reasonable range, Dicleli (2007) provided the isolated bridge with additional elastic device and discovered that the device is always effective to reduce the shear force of the isolator in moderate and strong earthquake regardless of how far the bridge from the rupture fault. In China, some scholars have also investigated the near-fault effects and its influence on bridge seismic responses. Jiang and Zhu (2004) studied the effectiveness and response characteristics of an isolated continuous bridge under near-fault excitations. The results showed that the product of speed pulse peak and its period is generally proportional to the seismic responses of isolated bridges. The LRB can still be applied to isolated bridges under near-fault excitations, but isolation effectiveness will not meet the expectation in several pulse-type ground motions.

The dynamic analysis model of isolated bridges and isolation effectiveness need to be verified by field testing and strong motion observation and other methods. With isolated bridges structure more and more widely applied, some scholars have done some researches on the model test and field testing to verify the security and effectiveness of isolated structures. Aiken et al. (1994) performed field testing to two isolated bridges over Walnut Creek, California, in order to gather field data and estimate the typical design hypothesis and analysis methods of isolated bridges and the reliability of the analysis model and technology of long-span curved viaduct with single pier. In 1994, the UC Berkeley performed a series of tests to a single pier of a four-span isolated steel-concrete composite beam with LRB, including fast testing, forced vibration testing and free vibration testing et al (Gilani et al., 1995). In 1996, the University of Buffalo NCEER performed a series of rapid field testing to an isolated highway bridge, using the hydraulic jack, cables and quick release device to conduct instantaneous release experiment of the whole bridge (Wendichansky, 1996). Chang et al. (2003) performed a field testing to one of the seven bridges newly built with LRB on the 2nd National Highway in Bai-Ho district in Taiwan. The tests including environmental vibration testing, forced vibration testing and free vibration testing showed that the dynamic properties and free vibration responses of isolated bridges can be acquired satisfactorily when considering non-linear characteristic of the LRB. Shen et al. (2004) performed the environmental vibration testing, free vibration testing and structural seismic monitoring in 1022 GIA-Yi earthquake to an isolated bridge with LRB to assess the design hypothesis and uncertainty in design and construction process.

The isolated bridge structure has a long natural vibration period, accordingly the superstructure will generally has large horizontal displacement under earthquake excitation, leading to a collision with adjacent structures, abutments or restrainers, even to a failure of isolation bearings. Thus the collision and displacement-limitation of isolation bearings are serious issues in isolated bridges. Characteristics of near-fault excitations and properties of isolated bridges may cause collisions between two adjacent isolated bridges, between beams and restrainers or the abutment (Padgett and DesRoches, 2008; Guo et al., 2009). The restrainer used before was mainly restraining the superstructure displacement in transverse direction, however, an issue that whether the displacement in longitudinal direction should be restrained or not is an new focus. Mitoulis and Tegos (2010) proposed a new external longitudinal restrainer, placing on the edge of rigid substructure, functioned by expansion joint and guiding plate and aimed to restrain the seismic movement of superstructure. The design hypothesis formerly assumed that the foundation was rigid and ignored the effect of the flexibility of site soil layers. However, the flexibility of site soil layers may strongly affect the responses of isolated bridges. The soil-structure interaction effect on isolated bridges is currently one of the most important research field. Olmos et al. (2010) investigated respectively about the effect of LRB devices designed upon current specification on bridge seismic responses in hard soil condition and medium soil

condition. The results showed that even in medium soil layers, the isolation system was effective.

In recent years, some scholars have studied the design method of isolated bridges. Scholars in Japan proposed the Menshin design method. This method is mainly about horizontal seismic isolation in longitudinal direction while restraining the transverse displacement of superstructure. The Miyagawa Bridge is designed according to this method (Lee et al., 2001). Tsai (2008) and Ryan and Hu (2009) investigated the seismic responses of partial displacement restrained isolated bridge and found out that due to displacement partially restraining, the composite damping ratio decreased, displacement partially restraining design is effective to reduce the shear force of pier but would increase the shear force of abutment and bending moment of superstructure. Cardon et al. (2009) investigated the simplified design method of bridges with strong non-linear seismic isolation system and considered two design methods: controlling the maximum horizontal shear force and controlling the maximum pier-girder displacement.

CONCLUSION AND DISCUSSION

Development overview and isolation devices for isolated bridge engineering are introduced, some new concerns including near-fault effects, soil-structure interaction, seismic collision and new design method in isolated bridge engineering are summarized and reviewed in this paper. From recent researches achievements, near-fault effect may impose notable influence on seismic responses of isolated bridge, seismic collision, unseating even failure of isolation bearing may occur in near-fault regions owing to large displacement of isolation bearing caused by pulse-type excitations. But isolation technology isolation technology may be feasible and effective even in near-fault regions and complex soil condition, but further researched on more appropriate seismic isolation bearing or hybrid control system and reliable design method are needed to provide help for prospective seismic effectiveness. More field testing, forced vibration testing and shock vibration testing should be performed on existing and future isolated bridges to confirm design hypothesis and uncertainty. Up to now, there is seldom strong seismic observation on isolated bridges all over the world, so more strong earthquake observation stations should be settled on existing isolated bridges to obtained actual seismic responses during strong earthquakes and examine current design method.

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The Advantages of the Analysis of Curved Box Beam Using the Grillage Method

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ABSTRACT

This paper first introduces the basic principles and stimulation methods of the grillage method, using such method to conduct stimulation analysis of a certain curved beam bridge—a ramp bridge of Jingkai Road in Beijing—as its engineering background. By using MIDAS/CIVIL, a software finite element method to analyze structures, a single beam model and grillage method model were built, and the comparisons and the analysis about deflection and stress among these models was conducted. The conclusion indicates that the calculation results obtained from the grillage method are closer to the values required in actual construction. The analysis revealed that the models based on the grillage method can reflect the mechanical properties and stress state more efficiently and accurately. Therefore, the grillage method is a very convenient and practical method for analyzing urban curved beam bridges, and it is verified that it can be used in the analysis of urban curved bridges.

Key Words: Curved beam bridges, grillage method, finite element, displacement and stress

INTRODUCTION

With the rapid development of modernization in China, the sustained growth and rapid expansion of economy, the escalating levels and accelerating development of road network construction, and increased updating of urban transportation, bridge construction has arrived at a new stage of rapid development, depicting great blueprints of “bridging the gap between the north and the south, and natural moats turning to be thoroughfares” everywhere.

Thus, various types of special-shape bridges have been widely applied in highway and urban overpasses, which have higher requirements related to line types at home and abroad. Curved beam bridges not only adapt to the requirements of terrains, surface features, and consistency with transportation lines, thus decreasing hydraulic resistance to piers and shipping better, but they also offer intuitive feelings of beauty due to the smoothness and fluency of their structural lines.

Therefore, curved beam bridges are becoming popular among engineers and

the design and study of curved beam bridges are increasing as well. Due to the complexity of curved beam bridge structure, its design is usually more difficult than the regular orthogonal bridges, and methods used to analyze orthogonal bridges are not sufficiently accurate to analyze special-shape bridges. Also, if some finite element software is used to handle it, it suffers from both low efficiencies and too many division units, making it difficult to be widely used and promoted by bridge designers. The most widely adapted method used is the grillage method. In this paper, the authors use the grillage method to analyze the structural characteristics and mechanical properties of curved beam bridges using MIDAS civil 2010 to establish single beam models to make comparisons and validations.

THEORETICAL ANALYSIS OF THE GRILLAGE METHOD

It is known that, under the action of load, the internal forces generated from inner and outer cross-sections in curved beam bridges may be different, i.e., the stress state inhomogeneity of the webs. But if the cross-section can be divided into several units by multiple longitudinal beams, the stress state of the cross-section can be presented more accurately. Because the cross-section of box girders can be regarded to consist of many flanged beams, the longitudinal beam units can be used to simulate flanged beams, and the lateral beam can be used to connect the longitudinal beam units and undertake the load together. Sometimes, for the purpose of convenient loading, some dummy longitudinal beam units are introduced to form the networks longitudinally and laterally.

Basic principles of the grillage method. The purpose of the grillage method is to divide the superstructure of a bridge into many small parts with vertical and horizontal grids where bending and torsional stiffness are concentrated in each part correspondingly—i.e., the vertical and horizontal stiffness in grillages can represent, respectively, the longitudinal, lateral bending stiffness and the torsional rigidity in actual structure. Therefore, it can be deduced that the resultant force of the longitudinal bending moment, the lateral bending moment, shear force, and torque should be equal to internal forces that the actual structures undertake.

The most basic principle of the grillage method is as follows: Under the identical action of load, the beam grillage models have the same torsion and deformation as the original models do. In addition, moments, shear forces, and torques generated in each grid unit represent the very resultant force generated in that part of the beam. Therefore, in the process of the building of grillage method models, different ways to divide the cross-sections will generate considerably different internal forces in each grillage. Therefore, it is very important to divide the cross-section of beam units properly for the calculation of the characteristics of the cross-section of each beam after the section is divided.

Principles of dividing grillages. The principles of dividing grillages are as follows: (1) In the process of bridge design, during the process of dividing the longitudinal grillages, it is favorable to try to make sure that the centroid axis of the overall cross-section coincides with that of other cross-sections as much as possible. Only in

this way, under the action of load, will the grillage models deform the same as the original models, which is in accordance with the principles of grillage method. (2) To ensure the accuracy of the internal forces in the main beam, spacing between each lateral grillage should not be too wide. In general, it is appropriate to take 1–1.5 meters as its spacing, and it also should add some lateral grillages in the position of midspan, $\frac{1}{4}$ span, $\frac{1}{8}$ span, diaphragms, and supports. To analyze the beam precisely, it is expected to densify the lateral grillages (decreased spacing of transverse grillage) in the positions where the internal forces change relatively largely to obtain a more accurate internal force diagram of the main beam. (3) Another critical question within the beam grillage method is to calculate the section characteristics of beam units. For ease in understanding, a case study using a real example to calculate the section characteristics of beam section in detail is presented.

CASE STUDY

Overview. An interchange ramp bridge of Jingkai Road located in the Beijing Fifth Ring Road is 25m +25 m +25 m three-span continuous curved beam bridge (radius of curvature is 48m), single-box, with dual-chamber box beam used as its the cross-section. The height of the beam is 1.5m, width is 9m, thicknesses of webs on both sides are 0.5m, roof thickness is 0.22m, plate thickness is 0.18m, and thickness of mid web is 1.0m without diaphragms on each midspan.

Calculation of section characteristics in each beam.

Division of beam unit. In grillage models, in accord with the basic principles of dividing beam units, the box beam can be divided into three pieces of longitudinal grillage to ensure the fineness of calculation. The positions of these three longitudinal beams correspond to the position of its beam webs, respectively, as shown in Figure 1. Some virtual longitudinal grillages are set on the flange of box beam, and there are 22 lateral grillages setting on each span with spacing of 1.0m. Also, the grillages located on supports have been appropriately densified.

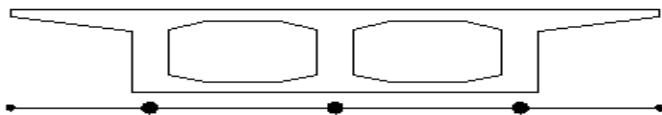


Figure 1. Grillage dividing box beam.

Flexural rigidity of longitudinal grid. In the analysis and calculation within the finite element software, the displacement can be obtained by solving compatibility equations, and then the internal forces can be deduced according to the deformation stiffness matrix. The correctness stimulation of grillage stiffness will directly affect the accuracy of its result.

When the longitudinal grillages were divided on the premise that the centroid axis of the overall cross-section coincides with the axis of other cross-sections, the authors divided the section in this example into three parts. All the dimensions of each part are presented in Figure 2. The bending stiffness of longitudinal beam is the

bending stiffness of its corresponding T-beam or flanged beam.

The value of inertia moment of component 1 and component 5 in the “virtual” side is the half of the value of inertia moment of the section of the cantilever.

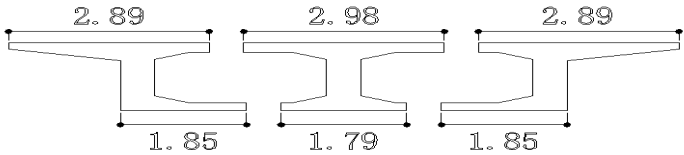


Figure 2. Division of cross section.

Torsional stiffness of longitudinal grid. The torque is a very important internal force in curve bridges, so it is necessary to accurately stimulate the torque stiffness. It is known that the shear flow will occur in the roof, floor, and web when the section undertakes an action of torque. Most of the shear flow passes through the floor, while a few of them pass through the mid web. When a section is under an effect of torque, it is appropriate to consider only the roof and floor plates in the rectangular sections of box beams, while the effects of webs and cantilever plates can be ignored. That is, when the torque stiffness is taken into account, the section of box beam can be simplified, as shown in Figure 3.

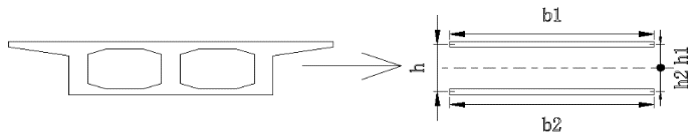


Figure 3. Torque of section model.

Ignoring the inertia moment of the roof and floor plates, the formula of torque inertia moment per unit length is:

$$i = d_1 h_1^2 + d_2 h_2^2$$

Since the position of the neutral axis is inversely proportional to the thicknesses of roof and floor plates, then:

$$i = \frac{h^2 d_1 d_2}{d_1 + d_2}$$

The constant of torque of grillage per unit length is:

$$i = \frac{2 h^2 d_1 d_2}{d_1 + d_2}$$

From the figure above, it can be seen that the even if the direction of the shear force loading on the web is opposite, some torque can be generated anyway, so it is necessary to consider the area of resisting shear, which is the area of the web of box beam section.

According to the formulas introduced above, the authors obtained the section characteristics of the longitudinal and lateral grillages from the real example, as shown in Table 1.

Table 1. Cross-Section Character.

Cross-Section Character	Longitudinal beam 1	2	3	4	5	Lateral beam
Bending moment of inertia(m ⁴)	0.000479	0.489	0.481	0.489	0.000479	0.1458
Torsion constant (m ⁴)	0.001587	0.2916	0.2916	0.2916	0.001587	0.2916
Shear area (m ²)	0.1747	0.4624	0.4624	0.4624	0.1747	0.0316

Comparative analysis of single beam method and grillage method. The dead load and live load of bridges were taken as examples to compare the differences of bending moment, shear force, and torque, which are based on the single beam method and the grillage method, respectively.

Contrast of Internal Forces in Single Beam Girders under Dead Load

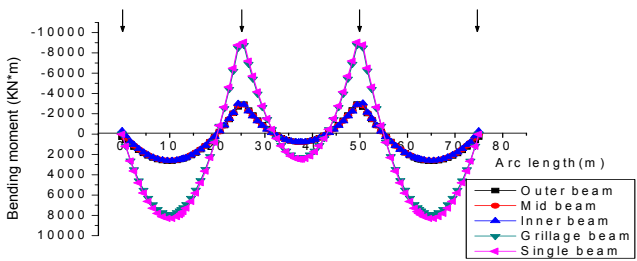


Figure 4. Comparison of moment of single beam grids under dead loads.

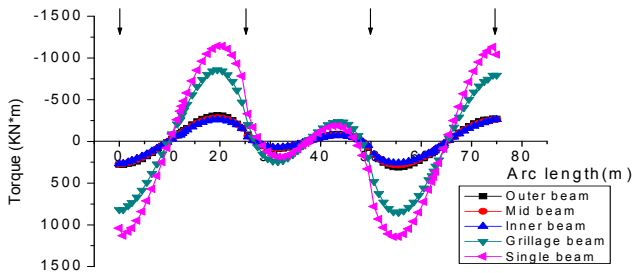


Figure 5. Comparison of torque of single beam grids under dead loads.

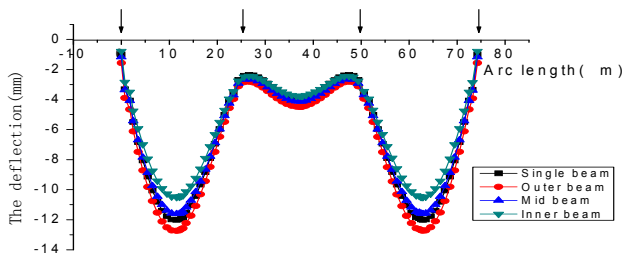


Figure 6. Comparison of deflection of single beam grids under dead loads.

1) As shown in Figures 4 and 5, bending moment diagrams that are based on the single beam method and the grillage method almost coincide—i.e., under dead load, the values obtained from the grillage are very close to the values obtained in the single beam units. This indicates that under the action of dead load, the differences of the bending moment undertaking on the sections are slight. The trend of stimulating moments based on both the single beam method and the grillage method fundamentally coincides with the actual condition of the original structure.

2) As shown in Figure 6, under the action of dead load, the differences of the torques obtained from the grillage method compared with the torques obtained from the single beam method are very large. This reflects a big difference in the effect of bending-torsion coupling resulting from the single beam method and the grillage method.

3) After a general study of the diagrams presented above, the grillage method can reflect the inhomogeneity of the stressed state of webs and deflections in curved beam bridges, which cannot be reflected using the single beam method. Since the length of the outer beam arc is greater than the length of mid beam arc is greater than the length of inner beam arc, under the action of dead load, the internal forces and deformations are generated on mid and inner beams. Such judgment is consistent with the trends shown in the diagrams, which means that grillage modeling is correct.

Contrast of Internal Forces of Single Beam Grids under Move Load

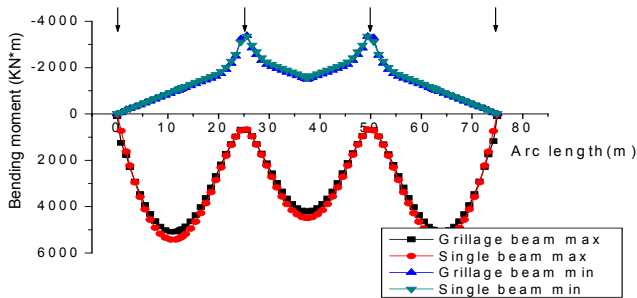


Figure 7. Comparison of moment of single beam grids under moving loads.

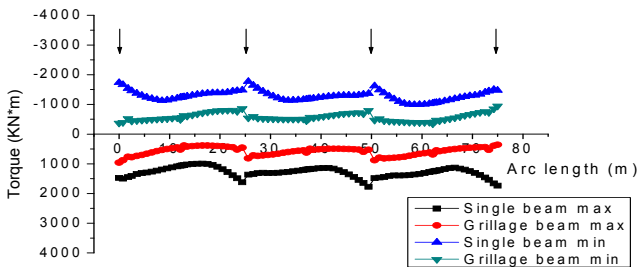


Figure 8. Comparison of torque of single beam girders under moving loads.

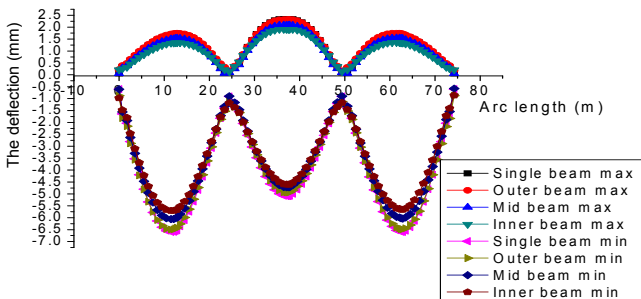


Figure 9. Comparison of deflection of single beam girders under moving loads.

- 1) From Figure 7, it can be known that, under live load, the value of internal forces obtained from the grillage method is very close to the values obtained from the single beam method. The maximum error of three-span midspan moment does not exceed 10 percent, which further verifies the principles of the division of longitudinal grillages and the method of calculation of bending stiffness of each section is correct.
- 2) From Figure 8, it can be known that, under live load, the value of torques loading on sections obtained from the grillage method is quite different from the values obtained from the single beam method.
- 3) From Figure 9, it can be known that, under live load, the value of the deflections on outer grillage are greater than the values on both mid and inner grillage, which is not revealed by the single beam method, and the deflection generated by the single beam method is slightly greater than that obtained from the grillage method.
- 4) After a general study of the diagrams presented above, under live load, the bending moment, shear force, torque, and deflection obtained from the grillage method are smaller than those obtained from the models of the single beam method. This is because in the grillage method the section is divided horizontally and longitudinally into networks, and the main function of the longitudinal beam is to undertake the load; then the lateral grillages are able to ensure that each longitudinal beam does not undertake the load alone. In addition, there is only one longitudinal beam undertaking the loading single beam method, so when vehicle loads take effect on the bridge, the internal forces and deformations in the models based on the grillage method are smaller than those based on a single beam bridge.

In summary, the grillage method is more comprehensive than the single beam method. In the grillage method, the section of curved beam bridge is divided into both horizontal and longitudinal grids, and the calculation of internal forces is different between the inner and outer side of the beam, which is consistent with the theory of the stressed state of a curved beam. The grillage method is relatively easier, and its results are relatively precise enough to satisfy the requirements of construction practice, so it can be adapted to calculate and analyze in the design of bridges.

CONCLUSIONS

Based on this research, the following conclusions on the grillage method were reached:

1) The grillage method can make the analysis of curved bridge sections more precisely, and it highlights the differences of the deflection and internal force between the inside and outside of the beam more than single beam method does, by which only the approximate internal forces can be calculated other than by conducting a further precise stimulation. In this way, the grillage method is more practical to satisfy the needs of actual construction.

2) By comparing the deflection, bending moment, and torque of some critical parts of models based on the single beam method and the grillage method, it can be known that the internal forces calculated from the grillage method are closer to the values required in actual constructions than those based on the single beam method.

3) Considering that the tasks of modeling and dividing analysis units in 3D solid models are complex and their calculation is always time-consuming, the grillage method is relatively easier and its results are relatively precise enough to satisfy the requirements of construction practice, so it can be adapted to calculate and analyze in the design of bridges.

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The Control Action of Full-Face Curtain Grouting Technology on Deformation of Tunnels

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ABSTRACT

For a wide range of reinforcement and high rates of water plugging, full-face curtain grouting technology is one of the major measures to improve soft surroundings and seal high-pressure water, especially to excavate underwater tunnels that lack a complex stratum owing to a large amount of groundwater. Because of high pressure, large quantities, and long cycles of injection, full-face curtain grouting technology has a great effect on tunnel excavation. This paper discusses the control action of full-face curtain grouting technology on deformation of a tunnel and recommends sensible preventive action.

Key Words: Full-face curtain grouting technology; deformation of tunnels; preventive action

INTRODUCTION

With the development of China's subway, various construction methods have emerged in an endless stream. These methods keep subway construction safe, fast, and smooth; full-face curtain grouting is one of them. Some scholars around the world have addressed it for a long time. Zhao (2009), with the study of Jiaozhou Bay's Qingdao harbor tunnel, which used full-face curtain grouting, indicated that a realistic appraisal of the tunnel's surrounding environment and a reliable grouting scheme were important for subway construction. Li (2011) simulated the construction scheme that controlled the settlement of building with the technology of full-face curtain grouting. The results of this study suggest that the technology improves the parameters of the strata, blocks the flow of groundwater, reduces the settlement of building, and ensures the safety of building during tunnel construction. Sun (2009) monitored surface deformation near the main shaft of

curtain grouting and analyzed the different monitoring points at different time intervals. The results suggest that the prediction accuracy with the average time interval sequence is the best. Zhang (2012) discussed the grouting materials of the Karst Tunnel. He indicated that the choice of materials in the Karst Tunnel grouting should make use of the improved single slurry, with other materials as supplements. Wang (2008), with the study of the choice of grouting parameters for single row holes, pointed out that the choice of grouting parameters for single row holes was reasonable in the construction of mine curtain grouting, and it would have a good grouting effect.

PROJECT OVERVIEW

The Guanhui intercity GZH-4 tunnel is 5.300km long and is located in the Dongcheng District, Dongguan City, and Liaobu Town. The task was undertaken by the China Railway Group's Limited Thirteen Bureau. The landscape is denuded hills and interdune trough. The terrain is volatile. The changes in elevation are between 6.2 and 133.3m. This tunnel successively passes through the Second Road of Guangming, the Avenue of Bayi, the road around the city, the superhighway from Dongguan to Shenzhen, four main roads of the southwest, the village of Shangtun, and the river of Huangsha. The main formation is silty clay, full-weathering mixed gneiss, strong-weathering mixed gneiss, and micro-weathering mixed gneiss.

The right line of the tunnel section is from GDK19+780 to GDK19+915, and the left is from GDZK19+780 to GDZK19+935. The left part passes through the alluvial grit VI soft surrounding rock. The stratum is weak and has a large amount of water gushing yield. To control the ground subsidence and water inflow, this project decided to adopt full-face curtain grouting technology to ensure the safety of the tunnel excavation.

GROUTING SCHEME DESIGN

When tunneling passes a section that is rich aquifer or fault fracture, the method adopted is full-face curtain grouting technology. The range of grouting is beyond the excavation contour line 4m (including the tunnel face). Along the direction of the tunneling, the grouting method is 17m for a cycle (the length of a cycle can be adjusted). When one grouting stage is completed, it will leave 3m as the sealing batholite for the next grouting cycle, as shown in Figure 1.

The full-face curtain grouting includes two components:

- One within 4m of the tunnel excavation contour line and the sealing batholite at the end of grouting circle. This part will form a monoblock to prevent water flowing into the tunnel.
- The other is the excavated face, which excludes sealing batholite. The

purpose is to remain excavated face stable.

PARAMETER DESIGN

Layout of grouting holes. The layout of the grouting holes radiates out from excavated face to the direction of excavation. The drill was evenly spaced in a circle to ensure grouting fully and not leaving corners. The grouting diffusion radius was 1.5m. A total of 101 holes were drilled during each cycle. The diameter of a grouting hole was not less than 110 mm, and the final hole was not less than 91mm. The orifice tube had a hot-rolled seamless steel tube. The length of the tube was 3m, the thickness 5mm, and the diameter $\phi 108\text{mm}$. The orifice tube was planted fast and had a good measure for borehole occluding.

Grouting material. In the water-rich section, cement water glass slurries were adopted. The volume ratio of slurry was set to C:S = 1: (0.6~1.0), the water cement ratio was 0.8:1 – 1:1, the modulus of water glass was 2.6 – 2.8, and the concentration of water glass was 30 – 40%.

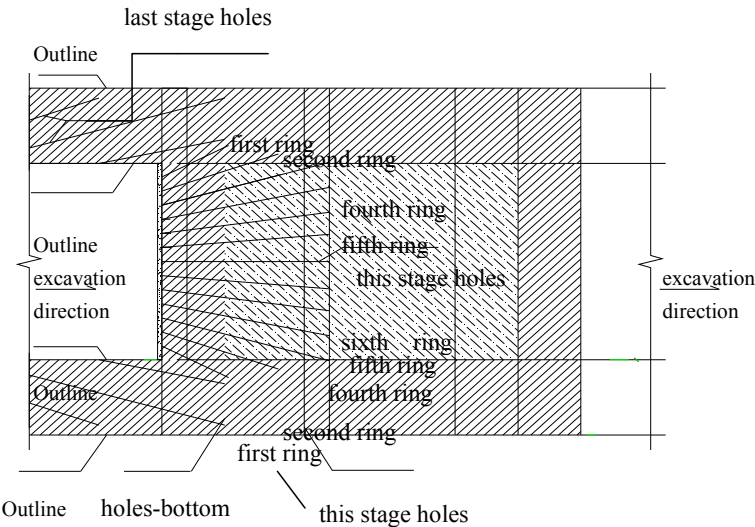


Figure 1. Longitudinal section of whole section curtain grouting.

End standard for grouting. The standard of end grouting for a single hole as that the grouting pressure increases gradually to the final pressure and will persist for 10min; the quantity of grouting was not less 80% of the design of the grouting volume. The inlet velocity as about 1/4 of the initial inlet velocity, and the water inflow of inspection hole as less than 0.2L/min.

The standard of end grouting for all grouting holes as that all the holes are

in line with the end condition of the single hole and the water gushing yield was less than $1\text{m}^3/(\text{d}\cdot\text{m})$.

Check and fill the grouting hole. When one grouting section was completed, it needed 2–3 holes to check the grouting effect. One method was to observe the filling of the core. The water gushing yield of the checking holes as less than 0.2L/min. The other method was a pressurized-water test. Under a pressure of 1.0MPa, the water gushing yield was less than 10L/min. The permeation coefficient was less than 10^{-5}cm/s , and the unconfined compressive strength was more than 0.8MPa. The construction parameters are shown in Table 1.

Table 1. Full-face Curtain Grouting Parameters.

Sequence	Parameter name	Values
1	longitudinal	17m
2	radial	working face and beyond outline 4m
3	diffusion radius	1.5m
4	pressure	0.5–1.5MPa
5	diameter of hole	Φ 91mm
6	velocity	5~110L/min
7	final hole spacing	2.5m
8	style	progressive segmented grouting
9	orifice tube	L=300cm, φ108mm

CONTROL ACTION OF FULL-FACE CURTAIN GROUTING TECHNOLOGY ON DEFORMATION OF TUNNEL

Full face curtain grouting is a continuous porous grouting technology. The slurry is pressed into the stratum by pneumatic or hydraulic equipment. The slurry flows in the pore or fracture of stratum and condenses into a certain thickness stone body that is continuous. The stone plays a role of reinforcement layer. At the same time, the injected soil whose permeability coefficient has increased improves the impervious ability.

In the process of full-face curtain grouting, when the grouting pressure exceeds the bearing capacity of the strata, it will create hydraulic fracturing to the original formation, and the slurry will fill the fractures and diffuses outward to the original formation. Because of the high grouting pressure, the large grouting volume, and the long grouting cycle, the full-face curtain grouting will produce great perturbations to the original formation. The perturbations cause a large variation of stresses in the embryonal structure of a stratum. The variation of stress will be released when the tunnel is excavated, and it has a great impact on the deformation of tunnel.

Control action of full-face curtain grouting on the excavated face. During grouting, the working face had long-term observation. With the increasing of grouting volume, the working face experienced different levels of bulging and even cracking, which caused grout gushing, as shown in Figure 2.

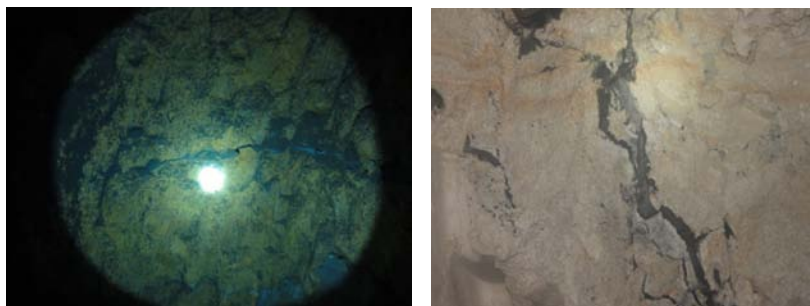


Figure 2. Swelling cracking.

The most likely reason for this was that the full-face curtain grouting had some characteristics such as high grouting pressure, large grouting volume, long grouting cycle, etc. Under high pressure, the slurry would cause high-intensity extrusion to the soil, so the porosity decreased and the density of the soil increased, and finally the working face generated the phenomenon of bulging. In the process of the full-face curtain grouting construction, the bulging phenomenon is very common. It is important to ensure tunnel face stability and then avoid the loss and casualties.

The effective measure to deal with the bulging phenomenon is to increase the reserved thickness of the tunnel face, reasonably controlling the grouting pressure and grouting volume and mesh reinforcement on the tunnel face. The most important is to conduct a reasonably accurate analysis of the initial conditions and pre-estimate the changes of stress after grouting and then formulate relative treatment measures to avoid cracking.

Control action of full-face curtain grouting on the surface subsidence.

Observation method of surface subsidence. The instruments of surface subsidence for observation were WILD-N3 precision level and invar rods. The measurement accuracy was II deformation measurement, which was set by *Specification for Underground Railway, Light Rail Traffic Engineering Measurement* (GB50308-1999). Technical requirements and indexes were executed according to the requirements of the standard. Before excavation, it should bury the reference point and set the leveling network beyond the ground deformation, which can keep stable for the long-term.

The times of observation should be increased properly for the first observation. Generally, it will require taking the data 2–3 times as initial readings.

The monitoring frequency of monitoring points was that:

- When the distance from the excavation face to the observation target was less than $2B$ (B is the span of the tunnel), the monitoring frequency was once or twice a day.
- When the distance from the excavation face to the observation target was less than $5B$, the monitoring frequency was once every two days.
- When the distance from the excavation face to the observation target was more than $5B$, the monitoring frequency was once a week.

The initial readings of the monitoring points should be gained within 24 hours after the last cycle, but before the next, and the distance from the excavation face to observation target should be less than 2m.

Control action of full-face curtain grouting on the surface subsidence.

A typical section was intercepted to analyze the changes of the surface subsidence value. The surface subsidence curve graph is shown in Figure 3.

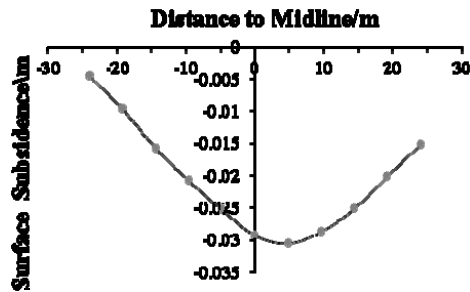


Figure 3. Surface settlement curve.

From the overall trends of the settling tank, the curve shows the approximate theoretical normal distribution. The surface subsidence basically was centrosymmetric distribution along the tunnel, and the settlement was controlled to the extent permitted. Given all this, the full-face curtain grouting technology met the requirements to strengthening the stratum.

In the process of grouting, some monitoring points showed bulge. The reason was that the slurry under high pressure played a role of compaction and fracturing to the stratum. It is suggested that surface subsidence was the joint result of bulge and subsidence, but, for the most part, it was settlement.

The way to control surface bulge or subsidence is to take a close look at the whole process of grouting and conduct a sufficient analysis and systemic argumentation of the initial structure of stratum. These measures could make the grouting pressure and grouting volume be controlled reasonably and then the surface subsidence can be controlled within the scope allowed.

Control action of full-face curtain grouting influence on horizontal convergence of the tunnel.

Observation method of horizontal convergence. The method of excavation for this project was Center Diaphragm. The changes of clearance were measured by convergence devices.

Mechanism analysis of full-face curtain grouting control action on horizontal convergence of the tunnel. A typical section was intercepted to analyze the changes in the horizontal convergence value of the tunnel. The horizontal convergence curve graph is shown in Figure 4. The curve was drawn based on the changes of horizontal convergence value.

As can be seen in the graph, the full-face curtain grouting caused a great disturbance to the initial stratum of the tunnel. In the initiation phase of grouting, the curve was volatile, and the amplitude was very large. The fluctuation switched from positive and negative. But overall, the fluctuation moved towards balance; in the late grouting, the changes of horizontal convergence tended toward stability and overall converged to near zero.

The full-face curtain grouting could strengthen the stratum and prevent water leakage. The reason is that slurry flowed in pores or fractures of the stratum and quickly condensed into a certain thickness cement-filling body that was continuous. In the flowing process, the slurry played a role of compaction and fracturing, which destroyed the original structure of the soil particles and made them restructure. From a macro perspective, this is the disturbance of the stratum. Specifically, the disturbed soil will absorb moisture from the soil around. In the short term, excess hydrostatic pressure showed a sharp increase and made the effective stress of the soil decline sharply. According to the theory of shear swelling, the structural strength of stratum would reduce further.

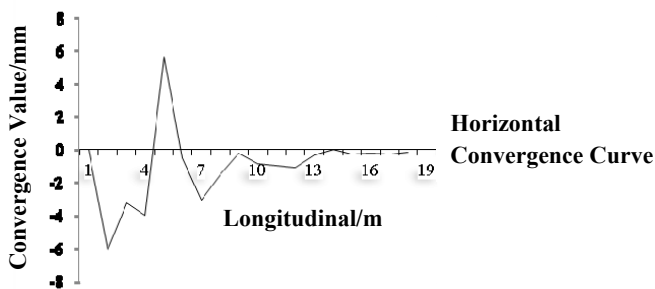


Figure 4. Curve of horizontal convergence change.

The influence on horizontal convergence was mostly because the slurry under high pressure destroyed the original structure of the stratum and the original crustal stress was reassigned. However, the influence played a major role in the early stage of grouting. In the late grouting stage, solidified slurry bore part of the crustal stress and made the stratum structure reach a new balance of force.

Therefore, the grouting technique reduced the influence on the horizontal convergence of the tunnel.

The influence on the tunnel can be controlled by appropriate technology. On the basis of geological exploration results and the geometric physical parameters of strata, the relative balance between grouting pressure and the lining strength was balanced and made them remain in dynamic balance. This can minimize the disturbance effect on the rock.

DETECTION OF GROUTING EFFECTS

The main test method of grouting effect was to drill an inspection hole and core. According to the analysis of the actual data measured on the construction site, the coring rate met the design requirement. Core images are shown in Figure 5. The images show that the bond strength of the rock stratum was greatly improved and the integrity of the cores was good. There was no water outflow from the inspection hole. This reached the grouting requirement.



Figure 5. Core sampling.

CONCLUSIONS

(1) Full-face curtain grouting technology has proven to be one of the major measures for improving soft surroundings and sealing high-pressure water. However, because of its limitations, it could cause accidents easily, and the economic benefits need to be optimized.

(2) The influence of full-face curtain grouting technology on tunnel deformation is mostly because the slurry under high pressure destroyed the original structure of the stratum and the original crustal stress were reassigned. However, the influence played a major role in the early stage of grouting. In the late grouting stage, solidified slurry bore part of the crustal stress and made the stratum structure reach a new balance.

(3) Carrying out strict controls on the excavated face, surface subsidence, horizontal convergence, and monitoring measurement to the construction site are key steps to ensure the success of grouting.

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Technical Measures of Shield Tunnel Construction in Alternating Hard and Soft Rock Strata

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ABSTRACT

Subway running tunnel construction is rather difficult when using an earth pressure balance shield in alternating hard and soft rock strata for problems of dealing with full-face hard rock, upper-soft lower-hard ground, full-face soft rock, and the interface of hard and soft rock. In response to these technical problems, taking measures of theoretical analysis, indoor tests, and combined with engineering practice, we studied the three follow matters: construction parameters for shield tunnels in different strata, the building settlement law, and countermeasures and construction technology for shield tunnels in complex strata. The main conclusions are as follows: (1) shield tunnel construction is demanding of a shield machine in alternating hard and soft rock strata such that the shield machine should have the ability to drive in hard rock and soft soil, preventing spewing, overbreak, etc.; (2) technical countermeasures and construction parameters of different strata are presented by cutting tools, driving mode, driving parameters, etc., and (3) all measuring points are affected by secondary disturbance when the rear shield passes buildings and the sedimentation value increases, which accounts for 45–66% of the final settlement.

INTRODUCTION

Subway running tunnel construction is rather difficult when using an earth pressure balance (EPB) shield in alternating hard and soft rock strata for problems of dealing with full-face hard rock, upper-soft lower-hard ground, full-face soft rock, and the interface of hard and soft rock. The measure that tunnels excavated by the mine tunneling method and segments assembled by a shield machine is often adopted in continuous hard rock strata driving more than 100m length (Vinai 2004). When the method that combines underground excavation and shield is not available, shield technology can solve the problem of long-distance hard rock excavation (Darrag 1999). In 1995, Guangzhou Metro successfully constructed a shield tunnel in complex strata (Yuan 2004). Since 1999, Guangzhou Metro has constructed a large number of tunnels using shield technology and accumulated valuable experience in the aspect of complex strata construction (Zhu 2004). By optimizing the driving parameters, using reasonable configuration tools, and strengthening the ability of wear-resisting,

Shenzhen Metro line #5 solved problems such as excessive wear of tools, changing cutters frequently, and advancing slowly and made it possible to excavate in long-distance hard rock stratum by a shield machine (Song 2010).

The heat generated by friction of the mining face and cutting tools and high temperature sediment accelerate the tools' abrasion in hard rock strata construction with EPB (Zhu 2006). Shield cutting tools, cutterheads, and screw conveyors are worn worse in extremely hard rock. There is a great deal of technical difficulty in the aspect of shield driving in hard rock stratum and the alternating hard and soft rock strata.

ENGINEERING FEATURES AND DIFFICULTIES

The shield method is used in subway running tunnels. The left line and right line of the tunnels are constructed, the tunnel is 1012m, and the cover depth of the vault is 10–22 m. The ring width is 1.5m. The shield machines in both lines passes through the whole face of the hard rock stratum three times successively. The rock is mainly composed of slightly weathered granitic gneiss, whose average uniaxial compressive strength is 99.4 MPa, and the strength of some rock test pieces reaches 192 MPa. Shield machines pass through high strength hard rock, soft rock, and complex strata with these two strata alternating, mixing, and mutating. The shield tunnel underneath passes through 10 buildings with various foundation forms.

Through analysis of the above characteristics, construction technical difficulties are as follows: (1) the high strength of rock and long distance driving result in a slow driving speed and high temperature, which easily cause damage to cutting tools; (2) the shield tunneling machine working in the mutation between a soft and hard stratum will lead to problems of cutters easily wearing out (they are difficult to replace), the shield posture being hard to control, the driving speed becoming slow, mud caking, spewing, and ground surface subsidence; (3) the shield machine passes through the left and right line two times, one after another, below the adjacent structures, causing a secondary disturbance and subsidence, resulting in adjacent structures producing uneven settlements, incline, collapse, and other damage.

SELECTION OF A SHIELD MACHINE

The project selected a composite EPB machine to adapt to the complex geological conditions, and the configuration and function should be improved and strengthened in following aspects according to the characteristics of engineering construction:

(1) A high-strength panel-style cutterhead, which could install both the disc cutters and gear shaping cutters, should be chosen. The cutterhead should be adaptable to the soft soil and the hard rock stratum, and high-strength disc cutters should be configured to break hard rock and gear-shaping cutters to excavate soft soil. The cutters also should be convenient and timely to repair and replace. The specific configuration and design are as follows: choose a high-strength panel-style cutterhead that supports installing disc cutters and gear-shaping cutters simultaneously; the cutter's design ability of rock fragmentation should be 200 MPa; the cutting tools should be configured to 5 center cutters, 30 single disc cutters, 100 front cutters, 12 edge scrapers, 12 edge protection cutters; and the cutterhead opening rate should be 28%. The maximum thrust should be 36,100 kN, and the maximum torque should be 6,305 kN•m.

The people brake system should be configured to realize the function of entering into the warehouse under pressure for cutting tools' checking and replacement.

(2) The shield machine should be able to prevent spewing. The configuration and design are as follows: (a) the elevation angle of the belt conveyor should be controlled within 12° to reduce fall sediment resulting from the spewing; (b) a double gate system should be set in the sediment notch of the screw conveyor to improve the function of spewing prevention control; (c) there should be an automatic pressure maintaining system so the soil chamber maintains pressure and prevents water from collecting too much in the soil chamber, which could result in spewing.

(3) The shield machine should have the capability of overbreak. To prevent the cutterhead getting stuck or the shield machine jamming in the hard rock stratum, the shield machine should be equipped with over-cutters to improve its applicability.

(4) The shield machine should have the ability of sediment improvement. To avoid spewing and mud caking, the machine should be equipped with a foam system and water system to improve the sediment.

(5) The shield machine should open some holes for polyurethane injection to seal the shield body when the water is large and handle the shield machine in emergency.

CONSTRUCTION PARAMETERS

Parametric analysis on shield tunneling. The running tunnels pass through the whole hard rock stratum several times. Taking the section of shield tunneling as an example, during left tunnel mileage ZDK21+961.516 ~ ZDK22+382.466 (corresponding to the number of segments from 78 to 388), the main driving parameters are shown in Figure 1. Achievements from Figure 1 include the following:

(1) In sandy clay and completely decomposed granite stratum, a shield machine can be maintained at a high speed; driving speed decreases significantly in transitional areas from strongly weathered granite strata to slightly weathered granite strata; on the contrary, driving speed increases in another transitional area from slightly granite strata to the strongly weathered granite strata. More stable driving speed is controlled in the section where full-face hard rock lasts longer. When it is upper-soft, lower-hard ground, the speed is about 25 mm/min and is within 5~10 mm/min when it is full-face hard rock ground.

(2) When a shield machine works in a relatively weak stratum, a lower cutter rotation speed is suggested to reduce the disturbance to the rock, averaging about 1.2r/min; in hard rock, a higher cutter rotation speed of about 1.3r/min will decrease the cutterhead torque.

(3) In upper-soft, lower-hard, and full-face hard rock ground, when with the hard rock area increases, the cutterhead resistance gets bigger, so its torque increases accordingly, mostly in 1500~3500 kN•m. When the shield machine passes through the third part of the hard rock in the left tunnel where hard rock ground continues for a long distance, the cutterhead torque reaches 3000 kN•m, sometimes 4000 kN•m.

(4) When the shield machine is driving in upper-soft, lower-hard, and full-face hard rock ground, the total thrust force 10000~15000 kN. The total thrust force in upper-soft, lower-hard ground is bigger than the one in full-face hard rock ground. The shield tunneling must be paid attention so as to control the total thrust force strictly to prevent

the local damage of cutters and segments caused by the large thrust.

(5) In soft rock and upper-soft, lower-hard ground, the pressure of the tunnel face is at a high level, but in full-face hard rock ground it stays lower and, due to good self-stability, it is better to maintain a small soil chamber pressure. For example, in the hard rock ground of the left shield tunneling, the tunnel face pressure ranges from 0–0.5 bar.

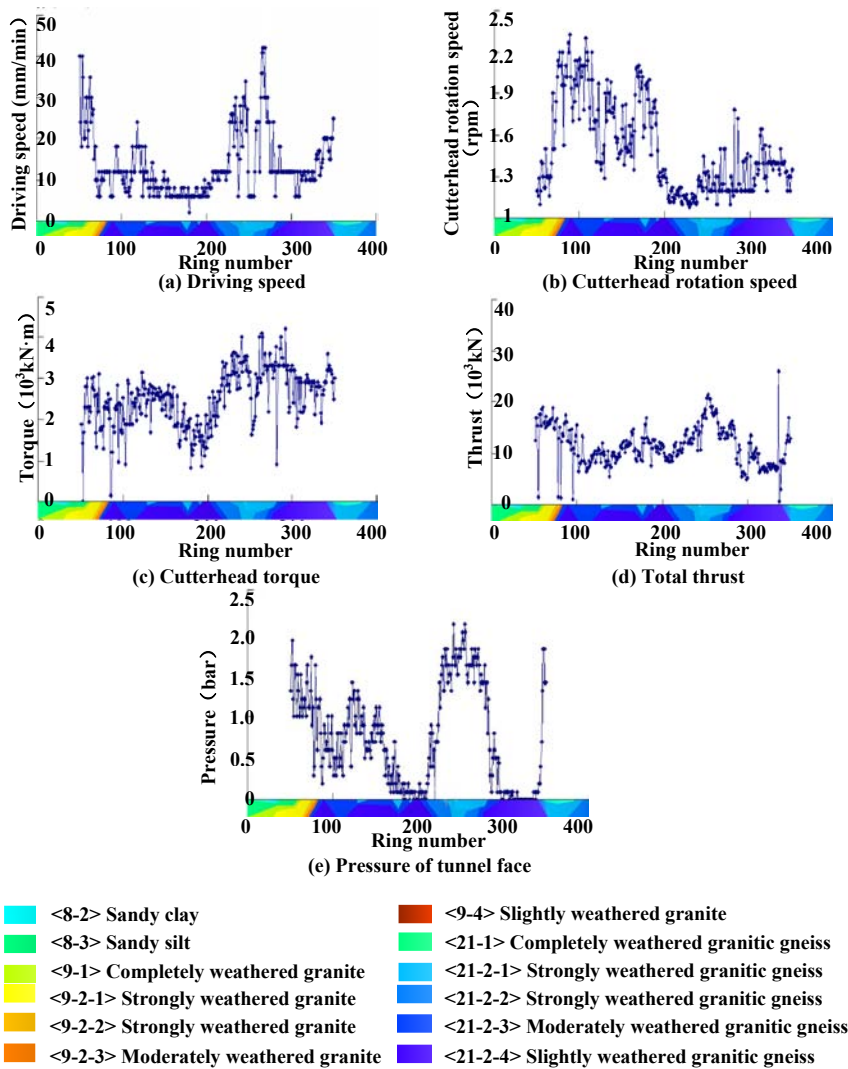


Figure 1. Curve of boring parameters and tunneling ring numbers.

Construction parameters for different strata. Through the parametric analysis on the shield tunneling above, combined with practical experience, relevant technical measures and driving parameters were studied and proposed in high-strength hard rock, upper-soft, lower-hard ground, full-face soft rock, and transfer strata. They are shown in Table 1.

BUILDING SETTLEMENT

Building foundation settlement is increased significantly when the antecedent shield is passing, and sedimentation is gradually stabilized after. When the rear shield passes buildings, all measuring points are affected by secondary disturbance and sedimentation value increases, accounting for 45–66% of the final settlement. The most significant effect on the buildings is the geological conditions of the shield section; when dominated by strong weathered granite gneiss, the largest settlement is 44.3mm; when it is slightly weathered granite gneiss, the largest settlement is 3.84mm. If the rock section includes three kinds of different degree rock and soil mass, the building settlement difference is relatively large, and the maximum of incline value is 1.69%. For example, on July 7–13, 2010, the left line shield undercrossed Red Dragonfly Kindergarten, and on August 7–12, 2010, the right line shield finished. The measuring point arrangement is shown in Figure 2, and the duration of settlement change is shown in Figure 3. From Figure 3, it can be seen that the building settlement has three obvious diachronic change stages. The building foundation settlement is increased significantly when the shield passes and sedimentation is gradually stabilized after through it; in the crossing process of right line tunnel shield, the buildings are affected by secondary disturbance and generate a larger settlement.

The application of the technical measures are presented for the technology difficulties of deformation controlling for double shield tunneling under the building in uneven soft and hard stratum condition.

(1) Geological investigation and stratigraphic analysis: The reinforcement should be prepared ahead for a poor building foundation and the easily sinking stratum. Before or after the shield getting through, it should be reinforced.

(2) Excavation mode: The open excavation mode is used in full-face hard rock; the earth pressure balance mode is used in soft rock and upper-soft lower-hard ground. Sediment improvement is conducted in tunneling and for ensuring that no extra sediment-out occurs when the earth pressure balance model turns to the air pressure balance model.

(3) Sediment-out quantity control: Excavated volume is controlled by adjusting the rotation speed of the screw conveyor in soft rock and upper-soft, lower-hard ground; the record of each ring's sediment should be made. The segment ring was 1.5 m wide in this project, and each ring's sediment quantity was controlled within 72.5 m³.

(4) Boring parameters control: Tunneling speed is controlled about 30 mm/min; the shield thrust is about 15000 kN, torque is 3000 kN•m, and cutterhead speed is about 1.3rpm, keeping even and uniform driving speed with small fluctuations of driving parameters and little cutterhead vibration.

Table 1. Driving Parameters and Technical Measures for Different Strata.

	Full-face hard rock	Upper-soft lower-hard ground	Full-face soft rock	Interface of hard and soft rock
Cutting tools	Choose high strength disc cutters. Check cutters and replace before hard rock. Check cutters every 10 rings, especially edge cutters. Tighten news installing cutters.	Check and replace cutters in advance before accessing upper-soft, lower-hard ground.	Choose high performance soft soil cutters.	Reinforce soil with poor self-standing ability or having adjacent buildings. Finish cutter replacement in full-face hard rock ground or before accessing upper-soft lower-hard ground.
Driving mode	Open mode (OM) suggested in full-face hard rock. Take water stop measures under air pressure when shield stop in high water quantity and sediment leakage area.	Transition from OP to earth balance mode (EBM) or from EBM to OP.	Adopt EBM or air balance mode on premise of sediment improvement.	Reduce or stop sediment-out before accessing soft rock. Build earth pressure and turn to EBM after sediment improvement. Bring down earth pressure after cutterhead touches hard rock, turn to OP.
Driving parameters	Control penetration within 20mm, cutterhead rotation speed 1.3–1.6 rpm, torque 1000–2000kN·m, and thrust is 15000kN according to torque.	Control cutterhead rotation speed 1.1–1.4 rpm, torque in 2000kN·m, driving speed about 30 mm/min, thrust in 16000kN, screw conveyer rotation speed with 3–8 rpm	Control thrust 11000–15000 kN, torque 1100–3200kN·m, cutterhead rotation speed about 1.3 r/min, driving speed about 40mm/min, soil chamber pressure stable 0.15MPa.	Adjust thrust to keep driving speed not too fast and reduce cutterhead rotation speed.
Vibration and voice		Adjust thrust and rotation speed when having greater vibration and noise. Stop and check if there is an abnormal condition.		Reduce cutterhead rotation speed and thrust if vibration is large in case of cutterhead jamming and cutter damage.
Grouting control	Adjust ratio of synchronous grouting to cut down setting time about 4 hours, improve consistency, and decrease segregation. Take secondary grouting.	Grout as much as possible to make sure stratum void can be filled timely under condition of no leaking. Take secondary grouting in time.	Control consistence in 10.5–11.0, grouting pressure about 0.3MPa, synchronize grouting/driving. Average grouting amount 6m ³ /ring. Take secondary grouting according to monitoring.	Increase amount of synchronous grouting and secondary grouting when stratum changes.

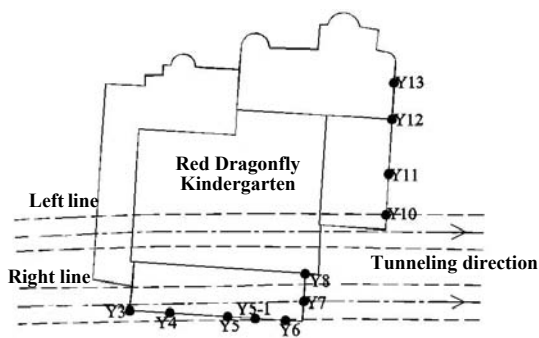


Figure 2. Distribution of observation points.

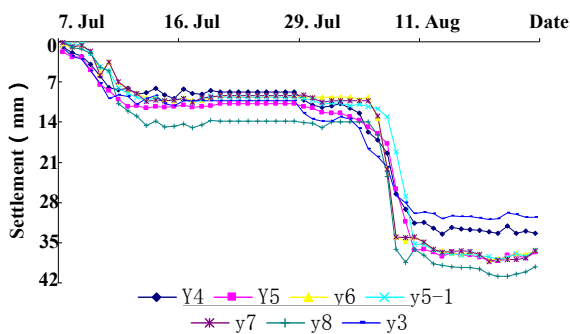


Figure 3. Settlement developing.

- (5) Cutter inspection and replacement: cutter inspection and replacement should be done before crossing the building, make sure that the shield machine can cross the building one time and not stay to replace cutters in the building section.
- (6) Grouting control: adjusting the synchronous grouting ratio and setting time, grouting volume is controlled in $6\text{--}75\text{m}^3$, the pressure is controlled in 3bar. Double liquid grouting behind segment is done at the same time, and the grouting pressure is controlled in 4–5bar to ensure rapid backfill density and consolidation. Synchronous and secondary grouting volume should be increased at the place having more sediment.
- (7) Monitoring control: aggrandizing the monitoring work of the ground and buildings. Placing a small catheter in the grouting behind the segment or ground grouting is determined by the sediment amount and monitoring.

CONCLUSIONS

- (1) Shield tunnel construction is demanding of a shield machine in alternating hard and soft rock strata. The shield machine should have the ability to drive in hard rock

and soft soil, preventing spewing, overbreak, sediment improvement, etc.

(2) Technical countermeasures and construction parameters of different strata are presented by cutting tools, driving mode, driving parameters, vibration and voice, and grouting control.

(3) All measuring points are affected by secondary disturbance when the rear shield passes buildings. The sedimentation value increases, which accounts for 45–66% of the final settlement. Geological conditions of the shield tunneling section are the most affective factors on buildings. The settlement is great when driving in weak rock and decreases significantly in hard rock.

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Nonlinear Dynamic Responses of Tunnels under Longitudinal Seismic Actions

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ABSTRACT

During strong earthquakes, the surrounding shield tunnel soil may enter the non-linear regime. When the springs and the dampers are utilized to simulate the soil-structure interaction, nonlinear properties of equivalent springs must be considered in dynamic analysis. This paper derived the nonlinear motion equations of tunnel segments under longitudinal seismic action. The characteristics of the nonlinear response of the shield tunnel segments under longitudinal seismic actions were analyzed. The influences of seismic P-wave velocity in soil and the parameter of tunnel segment bolting stiffness on internal force and deformation under non-linear soil-structure interaction were investigated. The numerical results showed that, if soil-structure interface enters nonlinear stress regime, the relative displacement between tunnel structure and soil will increase. This may lead to the increase in internal forces of structures which will produce stress concentration in structures.

INTRODUCTION

Many cities in China are located in high intensity seismic active area. In recent years, the seismic damage on underground structures has made seismic design of subway tunnels a very important issue, attracting more and more attention of civil engineers.

Seismic analysis of long underground structures can be divided into the cross-sectional seismic analysis and the overall longitudinal seismic analysis (Zheng and Yang 2011). Hashash and his co-workers (Hashash et al. 2001) developed the theory for the analysis of cross section of tunnel structure under seismic action, and discussed the effects of incident angles of seismic waves on stress-strain state of tunnel structure. He also proposed analytical formulae for calculation of soil-structure interface connection stiffness. For tunnel overall longitudinal seismic analysis, Chinese and foreign scholars also have conducted deep research. Liu (Liu 2003) used FEM to analyze the vibration of rod system in viscoelastic ground and analyzed the longitudinal earthquake resistance of shield tunnel with longitudinal joint of segments being simulated by joint element, and a formula being given to express the rigidity of longitudinal joint of shield tunnel. Zhang etc. (Zhang et al. 2012) analyzed the effects of tunnel bury depth, strata shear wave velocity, shear wave direction, elastic foundation coefficient, elastic modulus and the characteristic period of the overlying strata on longitudinal stress of tunnel lining. Xu (Xu 2013) discussed the effect of nonlinear foundation coefficient of subgrade, and bury depth of structure on the dynamic response of underground tunnel under longitudinal shear seismic action; he

further compared the influence of internal and external damping on the vibration displacement of tunnel, and confirmed that effect of external damping on structural vibration shouldn't be ignored.

Under seismic action, soil-structure interface generally experiences larger shear stress, which is caused by inconsistent deformation of two kinds of materials (Yin et al. 1994). The dynamic interaction between soil and tunnel structure demonstrates remarkable nonlinearity, hysteresis and cumulative deformation. For example, Zhang etc. (Zhang et al. 2004) performed nonlinear dynamic response analysis of Tianjin subway tunnel on seismic action by using finite element method. Jiang etc. (Jiang et al. 2005) analyzed the nonlinear response of shield tunnel when the longitudinal joint bolts enter plastic stress state; they mainly focused on the effect of the joint action of axial force and bend moment, bolt nonlinearity on the bolt stressing and deformation when equivalent stiffness method was used.

Based on the principle of equivalence deformation, Geng etc. (Geng et al. 2007) determined the stiffness reduction factors of shield tunnel under tension and compression, shear or bending moment, respectively. Their calculation indicated that the longitudinal tensile and compressive stresses induced by compression wave are the key factor in tunnel seismic design. Therefore the P-wave seismic analysis and research in the tunnel structure is of important theoretical and engineering significance. But at present Chinese and foreign seismic design specifications and regulations concerning the design under the action of longitudinal seismic waves are very crude and simple. However, when the longitudinal compression waves travels along the axis of underground structures, tunnel will perform longitudinal tensile and compression vibration which will cause the axial force in structure, as a result, bringing about stress concentration at the connections of the tunnel segment, at the junctions between tunnels and subway station structures. This paper considers soil-structure interface nonlinearity and discusses the longitudinal seismic response of the tunnel under the longitudinal compressive seismic wave. The effect of longitudinal wave velocity, the positions of segments and segment length on stressing and deformation of tunnel is investigated.

LONGITUDINAL SEISMIC ANALYSIS MODEL

The tunnel ring segments are connected by bolts. In order to simplify the analysis, this paper uses the single tunnel ring segment as the isolated body for analysis. The interaction between tunnel and surrounding soil is modeled by equivalent springs, as shown in Figure 1.

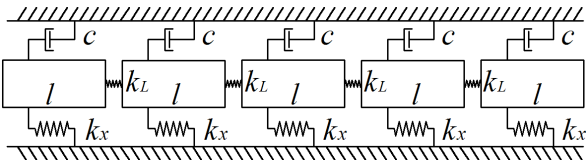


Figure 1. The simplified analytical model of longitudinal tunnel

The length of each tunnel ring segment is l , the outer tunnel ring cross section perimeter is L , shear coefficient between tunnel segment and soil foundation is k_x ,

the connection stiffness between segments is k_L .

SOIL - STRUCTURE INTERFACES DEFORMATION CHARACTERISTICS

Under the action of strong earthquake, tunnel structure and foundation may develop larger relative displacement, the interface between structure and soil may experience elastic-plastic deformation, at the same time, shear friction between foundation and structure is not linearly proportional to the relative displacement between structure and foundation. Thus when consider soil shear resistance to the structure, using the longitudinal shear coefficient k_x between the tunnel segments and the foundation to model the interaction between them can't meet the requirements

The relationship between the relative displacement between structure and soil structure shear force is nonlinear, namely the curve f_s is not linear function. The determination of f_s is an actual engineering problem, which can not be determined by theoretical method, but by experiments.

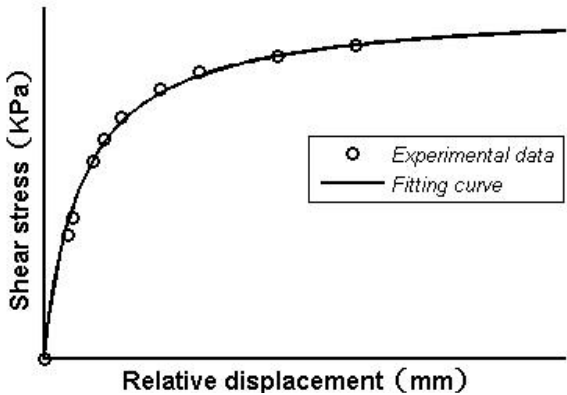


Figure 2. Nonlinear model of soil-structure interface contact

For the soil-structure interface constitutive relation, Clough (Clough and Duncan 1971) used the cassette direct shear apparatus to study the contact surface of the shear stress-displacement relationship, a hyperbola model was proposed to represent the relationship between shear stress and relative movement displacement, The obtained by him expression is: $\bar{\tau} = w_s / (a + bw_s)$, where a and b is the hyperbolic model parameters, which can be determined by conventional direct shear test. This constitutive model has been widely used.

TUNNEL LONGITUDINAL OVERALL SEISMIC ANALYSIS

Under seismic action, the tunnel structure dynamic response to ground motion is not directly caused by the direct application of seismic action on tunnel structure, but caused by the ground-motion (Liu and Du 2005).

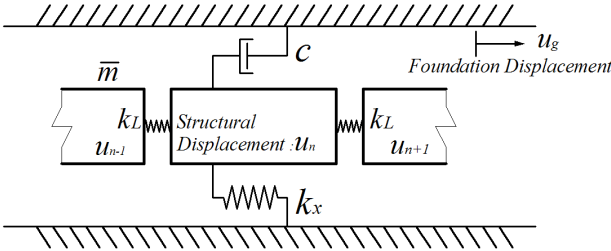


Figure 3. The simplified analytical model of n-th segment of tunnel

As shown in Figure 3, the mass of tunnel segment per meter is $\bar{m}$. Assume that surrounding tunnel soil evenly distributes along the tunnel axial direction, the soil displacement is u_g , the absolute displacement of $n-1$ -th, n -th and $n+1$ -th segment of tunnel are u_{n-1} , u_n and u_{n+1} , respectively. The relative displacement between the segment and foundation is $\Delta u = u - u_g$.

If the shear stiffness between soil and structure is k_{xl} under linear condition, and the friction resistance between soil and structure under nonlinear condition is f_s , then the equilibrium equations under linear and nonlinear condition may be obtained as follows, respectively.

Under linear condition, the equilibrium equation has the following form

$$\bar{m}l\ddot{u}_n + (k_{xl}l + 2k_L)u_n - k_L(u_{n-1} + u_{n+1}) = k_{xl}lu_{gn} \tag{1}$$

Under nonlinear condition, the equilibrium equation has the following form

$$\bar{m}l\ddot{u}_n + 2k_Lu_n - k_L(u_{n-1} + u_{n+1}) = L_p[f_s(\Delta u)] \tag{2}$$

Using the central difference method, the above mentioned equations may be transformed as follows:

Under linear condition:

$$\frac{\bar{m}l}{\Delta t^2}u_n^{i+1} = k_{xl}lu_{gn}^i + \left(\frac{2\bar{m}l}{\Delta t^2} - 2k_L - k_{xl}l\right)u_n^i + k_L(u_{n-1}^i + u_{n+1}^i) - \frac{\bar{m}l}{\Delta t^2}u_n^{i-1} \tag{3}$$

Under nonlinear condition:

$$\frac{\bar{m}l}{\Delta t^2}u_n^{i+1} = L_p[f_s(u_{gn}^i - u_n^i)] + \left(\frac{2\bar{m}l}{\Delta t^2} - 2k_L\right)u_n^i + k_L(u_{n-1}^i + u_{n+1}^i) - \frac{\bar{m}l}{\Delta t^2}u_n^{i-1} \tag{4}$$

Under nonlinear fitting the hyperbola function may be expressed as

$$f_s = \frac{s}{a + bs} \tag{5}$$

Substituting Eq. (5) into the equation (4) yields

$$\begin{cases}
 \dots\dots\dots \\
 \frac{\bar{m}l}{\Delta t^2} u_{n-1}^{i+1} = L_P l [f_S(u_{gn-1}^i - u_{n-1}^i)] + \left(\frac{2\bar{m}l}{\Delta t^2} - 2k_L \right) u_{n-1}^i + k_L (u_{n-2}^i + u_n^i) - \frac{\bar{m}l}{\Delta t^2} u_{n-1}^{i-1} \\
 \frac{\bar{m}l}{\Delta t^2} u_n^{i+1} = L_P l [f_S(u_{gn}^i - u_n^i)] + \left(\frac{2\bar{m}l}{\Delta t^2} - 2k_L \right) u_n^i + k_L (u_{n-1}^i + u_{n+1}^i) - \frac{\bar{m}l}{\Delta t^2} u_n^{i-1} \\
 \frac{\bar{m}l}{\Delta t^2} u_{n+1}^{i+1} = L_P l [f_S(u_{gn+1}^i - u_{n+1}^i)] + \left(\frac{2\bar{m}l}{\Delta t^2} - 2k_L \right) u_{n+1}^i + k_L (u_n^i + u_{n+2}^i) - \frac{\bar{m}l}{\Delta t^2} u_{n+1}^{i-1} \\
 \dots\dots\dots
 \end{cases} \quad (6)$$

Because $\Delta u = u_{gn}^i - u_n^i$, $f_S(u_{gn}^i - u_n^i)$ may be expressed as $f_S = \frac{u_{gn}^i - u_n^i}{a + b(u_{gn}^i - u_n^i)}$.

Substituting $f_S(u_{gn}^i - u_n^i)$ into Eq. (6) yields

$$\begin{cases}
 \dots\dots\dots \\
 \frac{\bar{m}l}{\Delta t^2} u_{n-1}^{i+1} = L_P l \frac{u_{gn-1}^i - u_{n-1}^i}{a + b(u_{gn-1}^i - u_{n-1}^i)} + \left(\frac{2\bar{m}l}{\Delta t^2} - 2k_L \right) u_{n-1}^i + k_L (u_{n-2}^i + u_n^i) - \frac{\bar{m}l}{\Delta t^2} u_{n-1}^{i-1} \\
 \frac{\bar{m}l}{\Delta t^2} u_n^{i+1} = L_P l \frac{u_{gn}^i - u_n^i}{a + b(u_{gn}^i - u_n^i)} + \left(\frac{2\bar{m}l}{\Delta t^2} - 2k_L \right) u_n^i + k_L (u_{n-1}^i + u_{n+1}^i) - \frac{\bar{m}l}{\Delta t^2} u_n^{i-1} \\
 \frac{\bar{m}l}{\Delta t^2} u_{n+1}^{i+1} = L_P l \frac{u_{gn+1}^i - u_{n+1}^i}{a + b(u_{gn+1}^i - u_{n+1}^i)} + \left(\frac{2\bar{m}l}{\Delta t^2} - 2k_L \right) u_{n+1}^i + k_L (u_n^i + u_{n+2}^i) - \frac{\bar{m}l}{\Delta t^2} u_{n+1}^{i-1} \\
 \dots\dots\dots
 \end{cases} \quad (7)$$

NUMERICAL EXAMPLES

In the following numerical example, we choose the following parameters for analysis: the elastic modulus of tunnel ring after reduction is $E = 4 \times 10^{10} \text{ N/m}^2$, the density of tunnel ring is $\rho = 2.5 \times 10^3 \text{ kg/m}^3$, the length of tunnel segment is $l = 1\text{m} \sim 1.5\text{m}$, the circumference of the outer cross-section of tunnel ring is $L_p = 20\text{m}$, the connection stiffness between tunnel segments is $k_L = 10^{10} \text{ N/m}^2$.

In the calculation the seismic waves are replaced by sine waves, the peak seismic wave acceleration is $a = 0.2g$, the sine wave period is $T = 0.35\text{s}$. Under the seismic action, the foundation displacement function is taken as:

$$u_{gn}(t) = \frac{a}{\omega^2} (1 - \cos(\omega(t - \frac{nl}{C_p})))$$

At the beginning the tunnel the foundation displacement is: $u_{g0}(t) = (a/\omega^2)(1 - \cos(\omega t))$, the maximum foundation displacement is $A_g = 12.18\text{mm}$. Here the time interval for calculation should be $\Delta t \leq (T_n/\pi) = 2\sqrt{(k/m)} = 0.0015\text{s}$, hence we choose $\Delta t = 0.0005\text{s}$.

The parameter for soil-structure interface friction function is chosen as follows: $a = 0.0000001223$, $b = 0.0001666$. The MATLAB codes are used to solve the linear and nonlinear problems. Under nonlinear conditions the equivalent secant spring stiffness between the soil and tunnel segments is taken as $k_x = 10^6 \text{ N/m}^3$, the

tangent equivalent spring stiffness is taken as $k_x = 8.18 \times 10^6 \text{ N/m}^3$. The numerical results are shown in Fig.4.

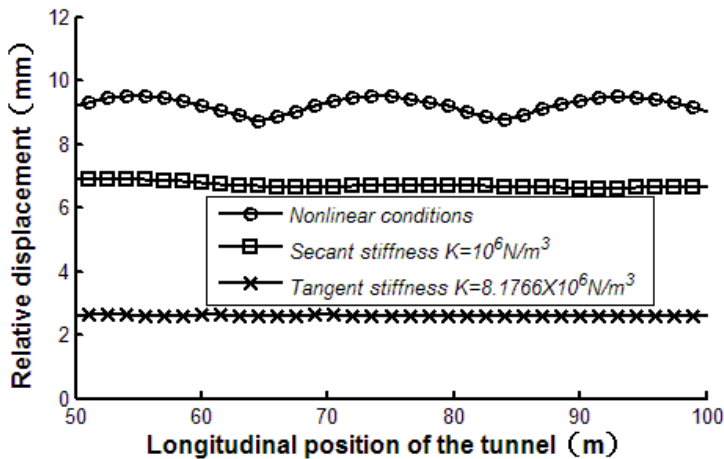


Figure 4. The dependence of relative displacement on the position of under different conditions

It is clear from Figure 4 that, when nonlinear effect is taken into account, the relative displacement between structure and soil becomes significantly larger, which tends to cause the slippage between soil and structure.

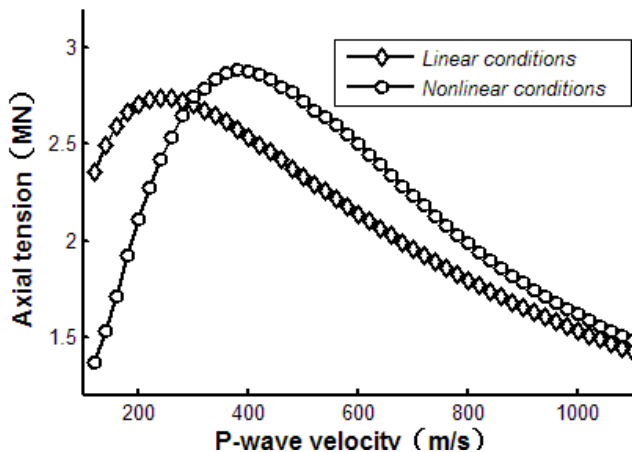


Figure 5. The dependence of the axial tension on P-wave velocity under linear and nonlinear conditions

Figure 5 shows the dependence of the axial tension on P-wave velocity under

linear and nonlinear conditions. As shown in Figure 5, with the increase of the compressional wave velocity in foundation, the relative displacement between the soil and structure decreases, the longitudinal tensile stress and compressive stress also decrease, and under nonlinear soil-structure interaction the tensile and compressive stress in structure are greater than that in the linear case.

Under the action of longitudinal compression waves, the axial tension and compression along tunnel take place, resulting in large tensile and compressive deformation. Under nonlinear conditions, the maximum tensile force in structure is 2.88MN, but under the linear conditions, the maximum tensile force in structure is 2.7MN. With the increase of P-wave velocity, the tensile and compressive stresses between tunnel segments decrease.

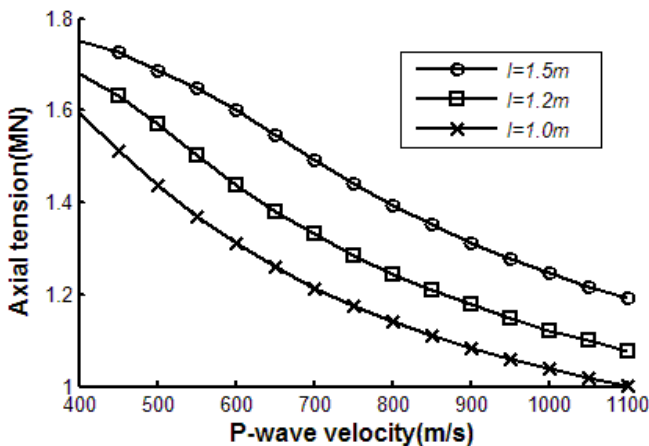


Figure 6. The dependence of the axial tension on P-wave velocity under different segment lengths

Figure 6 shows the dependence of the axial tension on P-wave velocity under different segment lengths. As shown in figure 6 the tunnel segment length has great influence on seismic performance of structures. With the increase of tunnel segment length, the tension and compression stress in structure increase gradually. Theoretically, smaller segment length is more conducive to the seismic performance of tunnel. However, if the economic problems and convenience for construction of tunnel are considered, the segment length of tunnel should be selected no less than 1m.

CONCLUSIONS

This paper presented a nonlinear dynamic response analysis of the tunnel under the action of the longitudinal seismic wave. The derived motion equations were solved numerically to determine the displacements of the tunnel segments. The effect of the longitudinal wave velocity, the soil- tunnel contact conditions and the tunnel segment length on the stressing and deformation was investigated.

Some conclusions from the above results can be reached:

- 1. If the nonlinear structure-soil contact characteristics are considered, the

- calculated relative displacement between structure and soil increases significantly, consequently the stresses in tunnel structure become larger, which tends to cause the soil-structure interface slipping.
2. Under the nonlinear foundation-structure interaction condition, the tension and compression stresses in tunnel segments increase. The internal forces in tunnel segments in hard foundations are smaller than that in soft ones, thus the seismic performance of tunnel structure in hard foundations is better than that in soft ones.
 3. Tunnel segment length affects the seismic performance. The longer the tunnel segment is, the greater the internal forces in tunnel structure are. The smaller the tunnel segment length is, the better the seismic performance of tunnel is.
- The above mentioned conclusions may provide references for the seismic design of underground structures.

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Application of the Geology Radar on the Health Detecting to the Highway Tunnels and Research on Data Analysis

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ABSTRACT

According to a real engineering of health detection of the Zhu Yong highway tunnels, this paper firstly introduces the proper parameters defining which should compare with other engineering and combine with the aims of detecting. Then on the base of post-treatment on the inspire results to the thickness and compactness of the left-right spandrel and the arch crown of the tunnel, referring to the construction design specification, it gives an evaluation about the lining thickness and the compactness, that is most parts of the lining thickness and compactness on the spandrel and the arch crown of the tunnel content with the requires of the construction and design, parts of that cannot satisfy it. At last the paper also discusses problems associated with using geological radar detection in tunnels and the process of radar signal denoising and line of determining.

Key Words: Road tunnels; geology radar; health detecting; data treatment

INTRODUCTION

With the current developments in tunnel detection, ground penetrating radar can find the location and types of diseases of tunnels accurately and is becoming more and more important.

Much research has been conducted related to ground penetrating radar in civil engineering. Dai (2000) summarized the main factors that affect radar exploration depth, resolution, and precision. Li (2000) summarized the contradiction between an antenna's frequency and exploration depth and pointed out that the higher the antenna's frequency, the shallower the exploration depth. Wang (1999) compared the effects of different parameters. Jia (2002) proved the reliability of ground penetrating radar detecting of tunnel liner depth based on several engineering examples. Li (2004) detailed how to arrange a line, choose an antenna, and set parameters when detecting tunnels. Feng (2004) arrived at the conclusion that the tunnel vault is the weakest part and should be paid great attention in tunnel detecting. In the long run, although ground penetrating radar is a great development in tunnel detecting, there are also some disadvantages. This paper is based on the engineering example of Zhu Yong

highway tunnel detecting and discusses the problems in the process of tunnel detecting and methods in data post-treatment.

ENGINEERING SITUATION

Bukit Mertajam tunnel is a separated tunnel; the left tunnel is 573m, and it is located in Zhe Dong low, hilly landform area. Rocks break into pieces, and its stability is weak. The surface water is a mountain stream with the characteristic of many ups and downs. The tunnel liner structure is a composite retaining structure. The initial lining thickness is level III surrounding rock and is 10cm; in the area of level IV, the surrounding rock is 15cm. A second line thickness in the level III surrounding rock is 30cm; at level IV, the surrounding rock is 35cm. Between the initial line and the second line is a waterproof and drainage sandwich.

DIELECTRIC CONSTANT

A dielectric constant is the main factor affecting the velocity of electromagnetic waves in ground-penetrating radar and detects the ability of material to store a polarization charge in an external electric field. A relative dielectric constant is expressed as a medium dielectric constant divided by the vacuum dielectric constant:

$$\varepsilon = \varepsilon_r \varepsilon_0 \quad (1)$$

Where, ε_r = relative dielectric constant, ε_0 = vacuum dielectric constant. The method in this detection to define the relative dielectric constant is the inverse method according to the formula:

$$D = V \times \frac{\Delta t}{2} \quad (2)$$

$$V = \frac{C}{\varepsilon^{1/2}} \quad (3)$$

Where, C= velocity of light, Δt = when the return trip, D= known depth of tunnel liner.

ANTENNA FREQUENCY

It is important to choose antenna frequency for detecting. A high frequency antenna has a high resolution ratio and high accuracy compared to a low frequency antenna; the fast energy attenuation leads to a shallow detecting depth. This detecting is aimed at getting the depth of the liner and the disease. If the necessary detecting depth is only 3m, a 500HZ antenna is suitable.

TIME WINDOW CHOICE

The time window (W) determines the maximum time limits during which the radar system can collect a single reflected radar wave. It indirectly influences the detecting depth. It can be calculated by follow formula:

$$W = 1.3 \times \frac{2d_{\max}}{v} \quad (4)$$

Where $d_{\max}$ = max detecting depth, v = velocity of electromagnetic wave.

SAMPLE RATE

The sample rate determines the time interval of the sample point, which comes from the reflecting wave, controlled by the Nyquist sampling theory. In most radar systems, the sample rate should be three times the antenna central frequency. Annan suggests that the sample rate should be six times the antenna central frequency.

$$\Delta t = \frac{1000}{6f} \quad (5)$$

Where Δt = sample rate, f = antenna central frequency.

MEASURING POINT DISTANCE

The measuring point distance is determined primarily by the antenna's central frequency and the underground medium's dielectric character; their relationship is as follows, according to the Nyquist theory:

$$n = \frac{75}{f\sqrt{\epsilon_r}} \quad (6)$$

Where n = measuring point distance, f = antenna central frequency, ϵ_r = relative dielectric constant.

When the lateral variation of the dielectric is not large, it can suitably relax the point distance to improve the working efficiency.

RADAR DETECTING DEPTH OF LINER

The antenna must be closely attached to the surface of the liner before detecting; the radar wave goes into the concrete liner via the antenna and will be reflected when it encounters the reinforcement, discontinuity, or interface of a different medium. It is received by the receive antenna. Based on the time of the wave incidence and reflection, it can calculate the length of the wave, and then the distance between antenna and the interface can be obtained.

When the antenna slips along the measure line, the time cure of every point measured can depict the time-profile imagine. If obtaining the reflect wave of all the relative points, it can directly obtain the depth of the liner by observing the different interfaces generated by the reflect wave.

MEASURE LINE DESIGN

There are three measure lines along the vault longitudinal direction: vault, left spandrel, and right spandrel as shown in Figure 1.

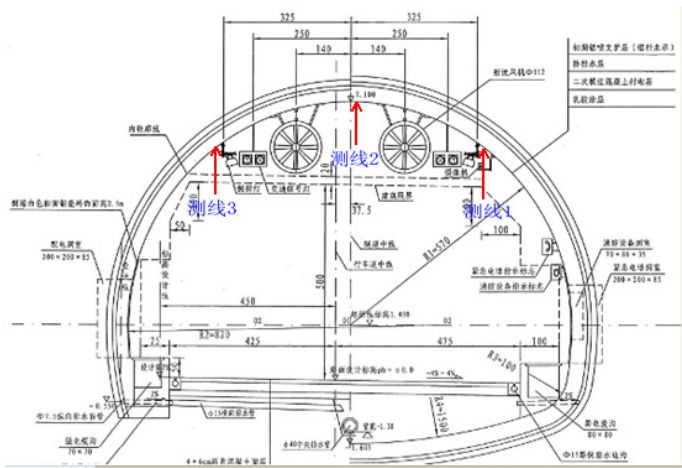


Figure 1. Measure line design.

DATA TREATMENT AND IMAGE EXPLANATION

Data treatment has two primary steps as shown in Figure 2. The first is correcting the pile number. Inevitably, there will be some errors in the data, so it is necessary to correct the pile numbers before data filtering. The second step is data filtering.

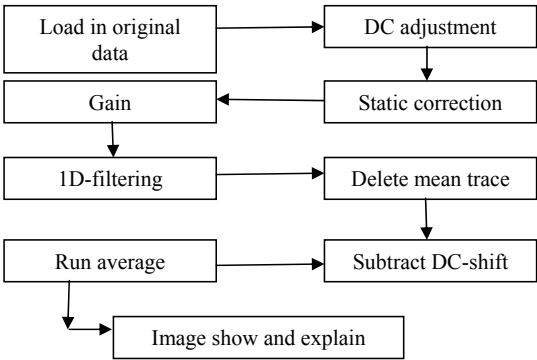


Figure 2. Tree of data treatment.

A DC adjustment is to remove the constant offset in the deep amplitude of the signal. The formula is:

$$X'(n, t) = X(n, t) - \frac{1}{M} \sum_{k=1}^M X(k, t) \quad (7)$$

In this way, it can effectively eliminate the stronger horizontal disturbances in the radar profile.

The amplitude comes from the depth reflect signal based on the length of the window. The aim of amplitude is to balance the energy of the available wave in the radar profile, and the means is let the wave of radar record multiply the gain weight function changed over time.

$$\tilde{y}(t) = P(t)y(t) \quad (8)$$

1D-filtering can activate the disturb signal and improve the signal-noise ratio. It also can pick up the response characteristics of the signal in the underground medium. When the available wave and disturb wave are similar, 2D-filtering is needed. After 2D-filtering, a filtering factor can be used in 2D convolution in the time domain, and a 2D spectral signal will have a forward and inverse transform in the frequency domain.

Figure 3 is an image of typical diseases. Understanding the image requires both professional knowledge of radar theory and engineering experience. Some typical diseases include the following:

- 1) Leakiness: The reflect signal from the interface is strong, the phase axis is discontinuous, the image is distributed in the zone but has a bad break and is messy.
- 2) Empty: The reflect signal from the interface is strong, the phase axis is alone, the wave is hyperbolic, and below it also is a strong reflect signal, but the difference in time travel between the two signals is large.
- 3) Void: The reflect signal from the interface is strong and the wave distributes a ribbon strip or triangle and always has serial reflections.

The Bukit Mertajam tunnel's left line is 590m, and most of the diseases occur in the vault; in the right and left spandrels, the typical diseases are empty and void, but the pass rate meets the requirement.

DIELECTRIC CONSTANT CHOICE

The dielectric constant directly affects the detecting depth. In engineering, the constant always is defined according to experience. Sometimes because the location is not suitable or the quantity of points is not enough, the constant measured may not be correct. In detecting, the constant should be at least 0.01.

MILEAGE AND PILE NUMBER

Problems mainly occur in two ways. The first is that the mileage is wrong. When the tunnel is long, the wrong mileage may be recorded. On the other hand, the ruler has a measurement error in the cure tunnel. So when detecting, the mileage should be

carefully noted and the rule measure should be a straight line. After detecting, the actual mileage and the design mileage should be compared to make sure there is no error. The second problem is that the mileage in the file is not suitable for the actual mileage. In detecting, checks should be made to ensure that the mileage in the file is suitable for the actual mileage.

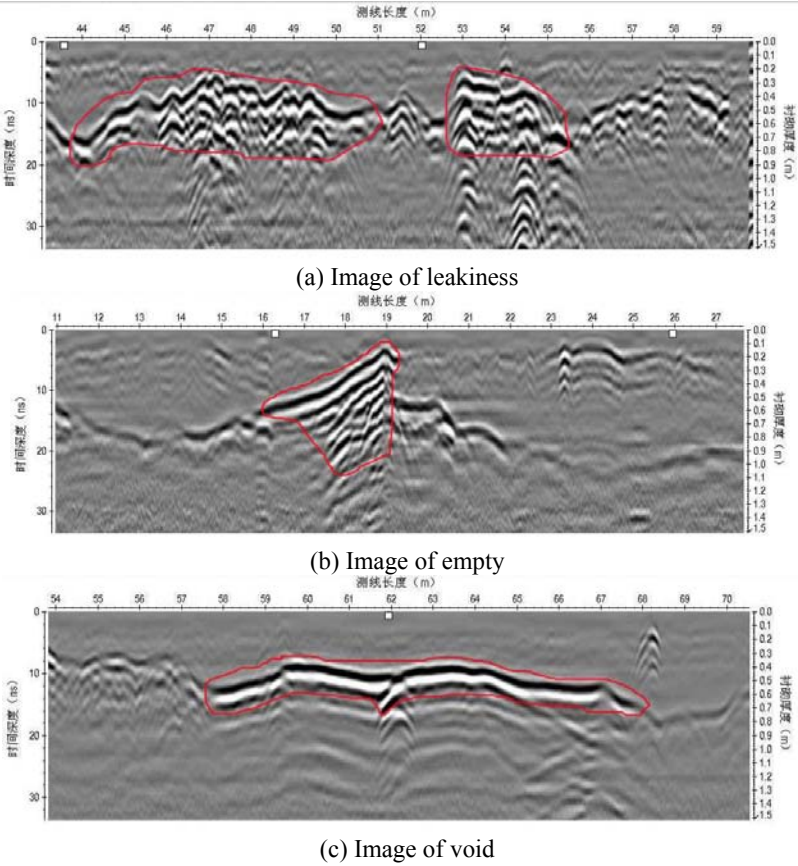


Figure 3. Image of typical diseases.

PROBLEMS WITH DETECTING

In detecting, because the instruments are not close to the surface of the liner or there is a low battery, the signal maybe so weak that it cannot get accurate information or not all measurements are recorded.

PROBLEMS WITH DATA TREATMENT

When loading in the pile numbers, the beginning and end numbers should be fit for the detecting direction. On the effects of some factors, the interfaces in the images of the detecting results maybe not clear, the post-treatment to the results should combine the geological situation and lamination.

CONCLUSIONS

In a detecting system, rational parameters must be chosen according to the basic theory of the process of detecting. All personnel should be professionals and should work carefully. Those who handle the data should have professional knowledge and experience in the practice of engineering.

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Structure Detection and Evaluation of Highway Tunnels Based on Geological Radar Detection Technology

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ABSTRACT

Combining highway tunnel testing and assessment with the application of geological radar technology in tunnel detection, key technologies of survey line layout, setting of acquisition parameters, and on-site testing are reviewed. Relying on field test data, lining thickness and density degrees of the left and right spandrels and the arch crown being safely used are more deeply researched. According to test results, whether the actual lining thickness of the spandrels and arch crown of the tunnel meets the requirements of design and specification are analyzed. Studies show that geological radar detection technology for tunnel nondestructive testing and evaluation is an effective method, and the correlation detection technique and analysis method have some reference value for similar tunnel structures.

Key words: GPR, tunnel, lining thickness, spandrel, arch crown

INTRODUCTION

With the continuous development of transportation, especially on mountainous area highways, the number of tunnels has increased. As a key part of highway construction, tunnels play an important role in shortening highway mileage, saving investment costs, increasing highway beauty, etc. However, the quality of tunnel supports is difficult to detect and can bring hidden trouble to traffic. In tunnel construction, common support quality problems include insufficient lining thickness, void zones that exist between the lining and surrounding rock, and slides that backfill. Operational diseases include water leakage of the lining, lining erosion, lining cracks, etc. These diseases affect the normal use of tunnels and tunnel lining durability. Therefore, a highly-efficient, comprehensive, and rapid detection method is needed to adapt to development, so that tunnel disease can be mitigated. Concrete thickness, compactness, and void of tunnel lining can be detected quickly by the GPR detection method. It not only overcomes the defects of the traditional incomplete detection methods (visual inspection and punch checks), but also uses high-tech means with high resolution and high accuracy and

is a fast and efficient method for nondestructive testing. It is widely used in tunnel detection.

WORKING PRINCIPLE OF GEOLOGICAL RADAR

The geological radar nondestructive detection method uses the reflected waveform difference of radar waves through the structure to analyze and process detection objects. It is generally used in the case where the detection area is large, detection objects are complex, requested detection precision is moderate, and detection speed is faster.

Geological radar uses pulse electromagnetic waves to detect the distribution of hidden media. The transmitting antenna of the geological radar transmits high-frequency, broadband, short-pulse electromagnetic waves to the tunnel lining. When the electromagnetic waves encounter the different dielectric properties of shotcrete and rock interface, some of the waves return. The receiving antenna receives the reflected waves and records their travel time. When the transmitting and receiving antennas along the object surface move synchronously, the profile images of internal medium of the object can be obtained. According to the received wave travel time (two-way travel time), the amplitude-frequency and waveform changes, and the internal structure of the medium, the target depth, and the shape feature parameters can be inferred (Figure 1).

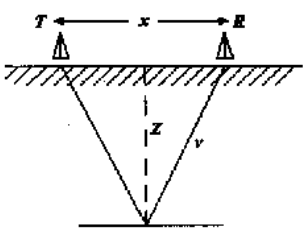


Figure 1. Principal chart of reflection detection.

Pulse wave travel time is calculated according to Equation (1):

$$t = \sqrt{4z^2 + x^2} / v \tag{1}$$

In the formula, the value of X is fixed in the profile detection; the value of V ($\text{m} \cdot \text{ns}^{-1}$) can be obtained using available data or measurement. The value of the target body depth can be obtained by the above formula.

The comparison of the relative dielectric constant of the tunnel lining and the surrounding rock determines whether stratification is “visible.” When there are defects, the contrast differences of the dielectric constant between the defects and the tunnel lining or surrounding rock make the defects visible, as shown in Figure 2.

500MB shielded antenna was chosen for the study. The acquisition parameters were as follows:

- Number of samples: 600
- Sampling frequency: 500MHz
- Superposition times: 8
- Time window: 37.28ns
- Trigger mode: time trigger, acquisition a wave every 0.1 seconds.

Detection essentials. To ensure the location of each detection point on the time section corresponding to the location of the actual detection mileage, a worker made a mark every 10m with chalk on the tunnel side wall. When detecting, the antenna in the corresponding line location close to the tunnel wall did continuous profile detection along the detection line direction. When the antenna aligned with a mark, the instrument operator input a signal to the instrument, and a tag was generated in the radar recording every 10m. When sorting data according to the mark, the first and final tag in the recording and the checked mileage in the work was indicated in the time profile chart to ensure the position accuracy.

Atlas analysis. This section of the detection image reflects the situation of the secondary lining of the tunnel, and the secondary lining as designed as a reinforced concrete structure. In general, due to the influence of secondary lining reinforced on electromagnetic waves, the steel arch of initial support was difficult to identify. But by deconvolution algorithm processing, the number of reinforcements can be seen clearly, and the number of lower steel arches can also be identified, as shown in Figure 4.

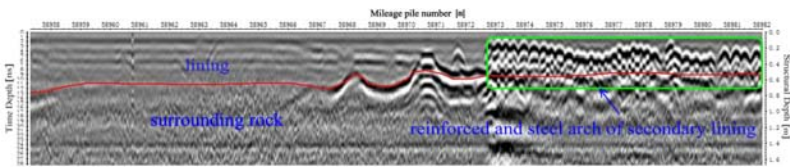


Figure 4. Wuzhuling Tunnel left line K58+964~K58+982 arch crown radar image.

This section of the detection image reflects the typical cavity. Because of the large difference between the dielectric constant of air and concrete, electromagnetic waves would generate a strong reflected signal in the cavity upper and lower interfaces (two different medium surfaces). At the same time, the diffraction signals will be formed at the lower interface and eventually will show as irregular curved multiple reflections, as shown in Figure 5. The input dielectric constant in the radar data post-processing stage is the dielectric constant value of the concrete. Since the medium in the cavity is air (there may be moist air) and the discrepancy of both dielectric constants is large, the velocity of the electromagnetic waves in cavity was difficult to determine. Therefore, the thickness of the cavity was not accurately calculated and can only be given a

rough value.

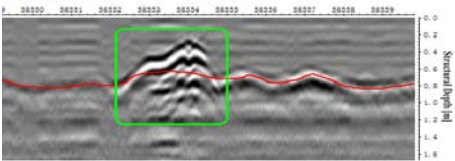


Figure 5. Wuzhuling Tunnel left line K58+550~K58+559 arch crown cavity region.

If the smooth blasting effect is bad, it can easily lead to a tunnel overbreak. To deal with this situation, rubble was used to fill the overbreak. Because of the existence of gaps between the rubble, the backfill often easily leads to leakiness between the lining and the surrounding rock or sometimes the cavity, so the litterly strong reflections are shown in the radar image, as shown in Figure 6.

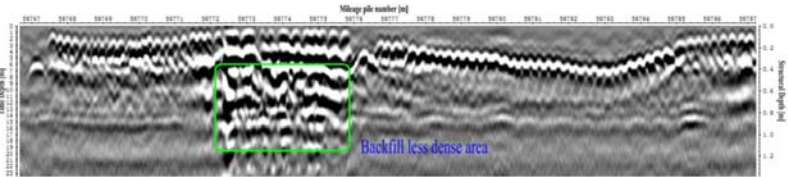


Figure 6. Wuzhuling Tunnel left line K58+767~K58+787 arch crown radar image.

The analysis images were obtained through image post-processing (filtering and stratification), as shown Figure 7. Finally, a lining thickness report is generated. In Figure 7, the red line represents the actual thickness of the secondary lining, and the blue line shows the design thickness of the secondary lining. In the structure depth coordinate direction, the position where the red line is higher than the blue line represents that the actual thickness of the secondary lining does not meet the design requirements.

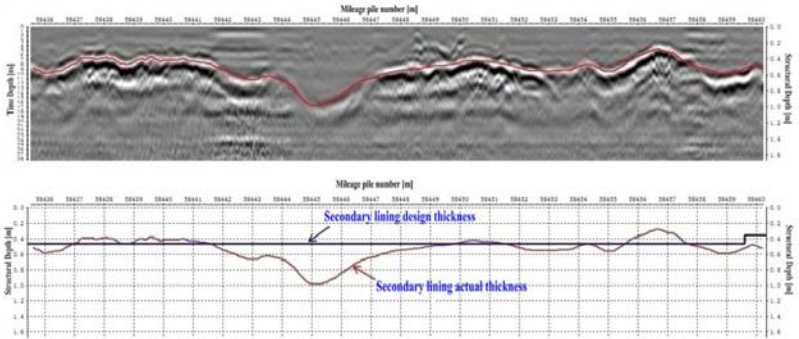


Figure 7. Wuzhuling Tunnel left line K58+436~K58+460 arch crown radar image.

TEST RESULTS OF LINING THICKNESS

A measuring point was extracted to evaluate the thickness at 0.5m intervals. According to the regulations in *Highway Engineering Quality Inspection and Evaluation Standards* (JTG F801-2004), the actual lining thickness cannot be less than the design value.

The length of the Wuzhuling Tunnel left line is 623m, the length of the three survey lines is 3×623m, and the number of measuring points is 3,770. The number of points whose thickness value was insufficient is 288, which accounts for 7.64 percent of the total measuring points. The cumulative length of the insufficient lining thickness is about 39.5m. (See Table 1 for details.)

Table 1. Main Section Statistics Where Lining Thickness of Left Line Is Lacking.

No.	Measured line positions	Start-stop mileage	Length (m)	Design thickness (cm)	Measured minimum thickness (cm)
1	Left spandrel	58708~58710	2	35	20
2		58785~58788	3	50	40
3	Right spandrel	58915~58919	4	35	25
4		58972~58975	3	50	40
5	Arch crown	58417~58419	1.5	45	35
6		58437~58441	4	45	40
7		58482~58490	8	40	30
8		58810~58813	3	45	25
9		58901~58904	2.5	35	25
10		58907~58914	7	35	20
11		58938~58940	1.5	45	30

The length of the Wuzhuling Tunnel right line is 645m, the length of the three survey lines is 3×645m, and the number of measuring points is 3,598. The number of the points whose thickness value is insufficient is 119, which accounts for 3.31 percent of the total measuring points. The cumulative length of the insufficient lining thickness is about 5.5 m. (See Table 2 for details.)

The pass rate of the lining thickness of the left line was 92.36%, so the measuring project is qualified. According to the type of lining sectional assessment, the pass rate of the lining thickness of K58+400~K58+444 section is 79.67%, K58+689~K58+738 section is 84.00%, K58+763~K58+804 section is 87.00%, and K58+965~K58+975 section is 82.67%, so these sections were unqualified.

The pass rate of the lining thickness of the right line is 96.69%, so the measuring project was qualified. According to the type of lining sectional assessment, the pass rate of the lining thickness of K58+736~K58+752 section was 88.00%, so this section was unqualified.

Table 2. Main Section Statistics Where Lining Thickness of Right Line is Lacking.

No.	Measured line positions	Start-stop mileage	Length (m)	Design thickness (cm)	Measured minimum thickness (cm)
1	Left	58448~58450	1.5	45	20
2	spandrel	58873~58875	2	35	25
3	Arch crown	58728~58730	2	35	30

CONCLUSIONS

(1) The primary support is the main load-bearing structure of the tunnel, which is built with the NATM and shotcrete bolt construction method. It is very important that the quality of the primary support is detected. The shotcrete thickness of the primary support of the tunnel can be detected by the geological radar.

(2) When detecting defects using geological radar, according to the thickness of shotcrete, the appropriate frequency antenna is selected. In addition to the thickness of the shotcrete being detected outside, the grille support in the shotcrete can be also probed and such defects as cavity and leakiness behind the shotcrete can be identified.

(3) To get accurate, reliable measuring data, in addition to calibrating wave velocity, alignment of the pile locations between the radar profiles and the actual tunnel mileage also is equally important so reliable data of the accurate position can be obtained.

(4) The detection speed of the geological radar is fast, and geological radar technology can apply to spot large area detection quickly, because of its fast detection speed and high efficiency.

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Shake Table Study of Bridge with Precast Post-tensioned Segmental Columns

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ABSTRACT

Precast, post-tensioned segmental bridge construction has been widely used in non-seismic regions in the U.S. for the objectives of accelerated bridge construction (ABC), higher quality control, less traffic disruption, and better workman safety. However, use of segmental bridges in moderate and high seismic regions is limited due to insufficient understanding of its behavior. This paper presents the results of a series of shake table tests on large-scale bridges with pre-cast post-tensioned concrete segmental columns. The seismic behavior of segmental columns was observed, segmental joint opening and closing were studied, and the self-center capability of segmental bridge was analyzed. This research is a U.S.-PRC international cooperative project established by the Multidisciplinary Center for Earthquake Engineering Research, University at Buffalo, the State University of New York, under contract with the Federal Highway Administration (Contract No. DTFH61-07-C-00020), and the Chongqing Communication Research and Design Institute, State Key Laboratory of Bridge Structural Dynamics, Chongqing, China, with the International Science & Technology Cooperation Program of China (Project No.: 2011DFA83300).

INTRODUCTION

As a promising approach of accelerated bridge construction, precast segmental bridge superstructures and/or substructures have been applied in numerous bridge construction projects in non-seismic or low-seismic regions, such as the Seven Mile Bridge, Sunshine Skyway Bridge, Mid-Bay Bridge, and John T. Collinson Bridge in Florida and the Louetta Road Overpass and U.S. Highway 183 Elevated in Texas. However, engineers still hesitate to use this construction methodology in high-seismic regions due to lack of understanding of its seismic behaviors, especially for columns, which is more critical to sustain the seismic load transferred from the superstructure

and the effects generated by ground movement in an earthquake.

During the last decade, several large-scale experimental studies on segmental bridge columns have been conducted. They include, but are not limited to, cyclic tests on tall and low precast, unbonded post-tensioned concrete segmental columns with steel jacket on the first segment (Hewes and Priestley 2002); cyclic tests on large-scale, precast, prestressed segmental slender concrete bridge columns (Ou, Wang et al. 2006; Wang, Ou et al. 2008); cyclic tests on large-scale, precast, unbonded post-tensioned segmental concrete bridge columns with normal and high-performance energy dissipation rebars (Ou 2007; Ou, Chiewanichakorn et al. 2007; Ou, Tsai et al. 2010; Ou, Wang et al. 2010); and shake table tests on one-span large-scale bridge models (Sideris 2012).

In previous experimental studies, important observations were obtained: a) severe concrete crush at the toe of the first (lowest) segment were found due to joint opening/closing and the strong impact force between segments if the concrete is not under enhanced confinement; b) comparing to the bonded prestress strand/tendon, unbonded post-tension strand/tendons can reduce the possibility of strand yielding; however, when the post-tension strands/tendons are put into the column wall, the dowel effect will yield the strand/tendon at the local place at the segment joints when large joint opening and sliding occur; c) shear key in between segments are necessary to resist the sliding and lateral force at the joints; and d) the energy dissipation bar can improve the energy dissipation capacity of the segmental column, but, in the meantime, reduce its self-center capability.

In light of the above observations, since 2009 Multidisciplinary Center for Earthquake Engineering Research (MCEER, University at Buffalo, The State University of New York) initiated a study on the dynamic behavior of bridges with precast unbonded post-tensioned segmental columns (strand placed in the center hollow of the column) and concrete shear key at the joint, and started to explore high performance concrete (HPC) for the segmental column connections. In 2012, a U.S.-PRC international cooperative research project was established by MCEER and CCRDI (Chongqing Communication Research and Design Institute, Chongqing, China), in which a series of quasi-static cyclic tests and shake table tests were performed on large-scale bridges with segmental columns. In this paper, the research on the shake table tests of large-scale bridge models with normal concrete segmental columns is presented.

SPECIMEN DESCRIPTION AND INSTRUMENTATION

The prototype structure was a five-span, box girder bridge with three interior spans of 30.48m and exterior spans of 22.86m, which is considered in Megally’s report (Hewes and Priestley 2002) (shown in Figure 1). The mid span of the prototype bridge was taken as the object of the test study.

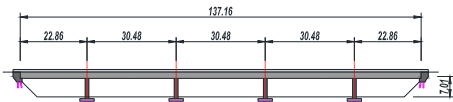


Figure 1. Elevation view of prototype bridge (m).

The experimental specimen was a large-scale (1:3.37) one-span bridge with a monolithic concrete superstructure and two segmental columns. The span length was 8.500m and the length of superstructure as 12.250m. The superstructure extended the column for 1.875m to represent the approximate inflection point at near spans (shown in Figure 2). The cross-section of the superstructure was a $3.000\text{m} \times 0.470\text{m}$ rectangle, with a $2.500\text{m} \times 0.220\text{m}$ rectangular notch inside. The length was 12.250m and the weight was 28 metric ton. The total weight applied on each column (superstructure plus column) was around 6% of the design axial resistance of the column. The substructure was composed of segmental concrete columns, cap beams, foundation blocks, and bearings. Each column consisted of four hollow segments with the size of $0.450\text{m} \times 0.450\text{m} \times 0.450\text{m}$ and a wall thickness of 0.110m. No continuous longitudinal mild steel rebar or energy dissipation bar passed through the joint. The column segments were connected only by post-tension strands. Concrete shear keys were designed and prepared for the alignment and to resist the shear force at the joints. The geometry of the column is shown in Figure 3. The designed post-tension force applied on each column is 375kN, which equals to 15% of the design axial resistance of the column.

Two types of bearing, non-seismic rubber bearing and fixed support (shown in Figure 4), were considered in this study. The purpose of using non-seismic rubber bearing was to simulate the practical project and allow the seismic load of superstructure to successfully transfer to the bottom of column, instead of isolating the seismic load by seismic bearing. Generally, the fixed support is not used in practical bridge project. However, in this study, the fixed support was used to generate the worst scenario, in which the super- and sub-structure were rigidly connected, and all seismic load generated on the superstructure transferred into the columns. In this way, the columns experienced the largest lateral load and the most impact during the shake table tests in the transverse direction.

Six seven-wire (diameter: 15.2mm) strands were used to post-tension the column segments, cap beam, and foundation block after 28-day of curing. All the strands were unbonded and put into the hollow of columns. Figure 5 shows the assembled bridge model on shake table.

The instrumentations in the tests included displacement transducers, accelerometers, strain gages, and load cells, which measured the joint opening and absolute displacement of foundation block, cap beam and superstructure, absolute acceleration at the shake table, foundation block, each column segment, cap beam, and superstructure, strain on the longitudinal and transverse reinforcement on the 1st segment, and the post-tension force in the strand, respectively.

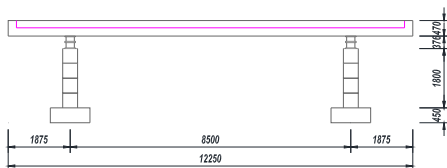


Figure 2. Side elevation view of bridge specimen (mm).

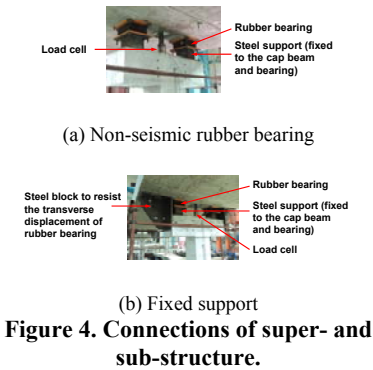
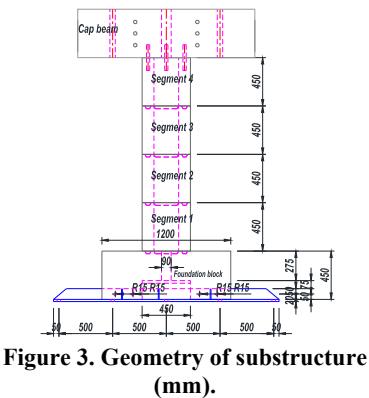


Figure 5. Bridge model.

SIMILARITY CRITERION

An elastic-gravity similarity criterion was used to analyze the seismic behavior of the bridge model in both the elastic and plastic stages. Table 1 shows the similitude ratios for the major physical quantities in the shake table test.

Table 1. Similarity Relationship of Physical Quantities in Shake Table Test.

Physical quantities	Dimension	Symbol	Value
Length	L	l	3.3697
Young's Modulus	$ML^{-1}T^{-2}$	E	1
Acceleration of gravity	LT^{-2}	g	1
Strain	$ML^{-1}T^{-2}$	σ	1
Time	T	t	1.8357
Displacement	L	u	3.3697
Velocity	LT^{-1}	v	1.8357
Acceleration	LT^{-2}	a	1
Angular velocity	T^{-1}	ω	0.5448
Mass	M	m	11.3549

EARTHQUAKE GROUND MOTION

The ground motions (GMs) used in the tests were selected mainly from FEMA P695 Table A-4A (Summary of Earthquake Event and Recording Station Data for the Far-Field Record Set) and Table A-6A (Summary of Earthquake Event and Recording

Station Data for the Near-Field Record Set). El Centro GM and Mianzhu GM (from Wenchuan Earthquake, 2008) also were selected. In total, five ground motions were used: El Centro GM (far field), Northridge GM (far field), Imperial Valley GM (near field without pulse), Cape Mendocino GM (near field with pulse), and Mianzhu GM (near field with pulse).

SeismoMatch v2.1 (software published by Seismosoft Ltd) was used to modify the ground motion to better match the target acceleration response spectrum (RS) and, in the meantime, remain their important frequency and extreme value characteristics. The maximum considered earthquake (MCE) response spectral accelerations used for collapse evaluation of Seismic Design Categories (SDC) B, C, and D were used as the target. The target RS, RS of matched GMs and their geometric mean value are compared in Figure 6. It should be noted that the time-axis is scaled by the time similitude ratio ($\lambda_t=1.8357$), and the starting point of the spectrum is normalized to 1.0g.

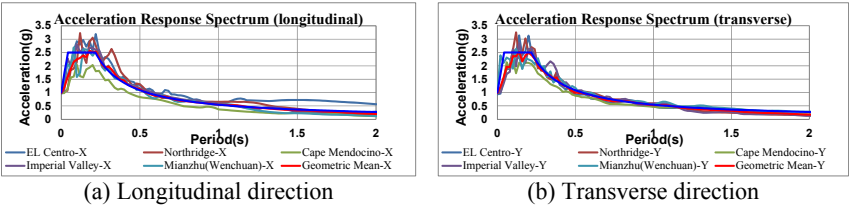


Figure 6. Comparison of target RS and RS of matched GM.

TEST PROTOCOL

As an exploratory shake table test, the bridge models were not designed to resist a particular earthquake at a particular intensity, such that they were tested under selected ground motions at an increasing intensity with different direction combinations. The bridge model was tested in two phases. The Phase I bridge was tested with non-seismic rubber bearings until the input peak ground acceleration (PGA) reached 0.5g, and the Phase II bridge was tested with fixed support until the specimens were damaged and lost their functions, or the displacement of the components exceeded the maximum measurement of the devices. It should be noted that in Phase I, the bridge was tested under the combinations of three directions, X, Y, and Z, while in Phase II, the bridge was tested only under the combinations of two directions, Y and Z. In between the tests, 48 seconds 3D-white noise (for non-seismic rubber bearing) or 2D-white noise (for fixed support) was used to determine the natural frequency and damping ratio of the bridge model.

TEST RESULTS

Figure 7 shows the maximum seismic response of the bridge model in Phase I and Phase II, which includes maximum acceleration and displacement at cap beam, joint opening between foundation block and the 1st column segment, and the maximum strain on the vertical rebar at the bottom of the 1st column segment. It shows that the

maximum acceleration response on the cap beam was 0.88g in Phase I and 1.70g in Phase II. The maximum amplification factor of acceleration response to PGA on cap beam was 2.1. The maximum displacement response on cap beam was 33mm in Phase I and 61mm in Phase II, which are equivalent to drift ratio of 1.83% and 3.40%, respectively. No residual displacement was found at the end of tests. When PGA was 0.4g, the strain at the bottom of the 1st segment exceeded 300 $\mu\epsilon$. Cracks were found at the corner of column's toe. The maximum strain at the column toe was 850 $\mu\epsilon$, and the concrete had already been crushed at this point. The maximum joint opening during the test was 9.1mm, which occurred at the bottom joint. However, not much joint opening was found at other joints. All the openings were closed at the end of tests.

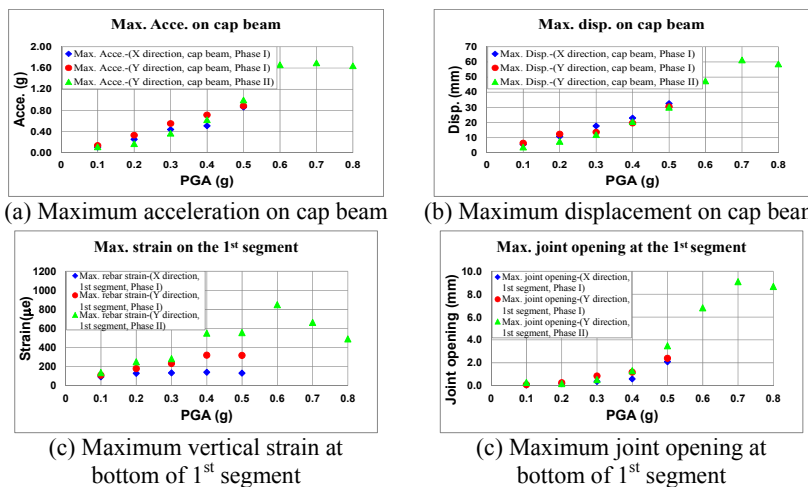
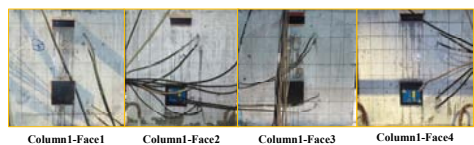


Figure 7. Maximum seismic response.

After Phase I (in which non-seismic rubber bearing was used and maximum PGA of input ground motion was 0.5g), only a very small region of concrete at two corners of the column toe cracked (which is not shown in this paper). Figure 8 shows the concrete crack, spalling, and crush at the 1st segment of the columns after Phase II (in which fixed support was used and maximum PGA of input ground motion was 0.8g). Concrete crack, spalling, and crush at the 1st segment were found, especially on the face perpendicular to the weak axis. However, no rebar yielding was found at the plastic hinge. From the system identification analysis, which was introduced later, this concrete loss induces an 18% decrease in the 1st mode natural frequency and a 10–20% increase in the damping ratio (in Phase II).



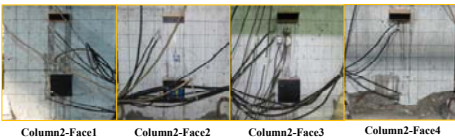


Figure 8. First segment of columns after shake table test.

For the dynamic test, fundamental frequency and damping ratio were the most important indices to evaluate the properties and damage of specimens. In this research, self-power spectral density function was used to determine the fundamental mode, and half-power bandwidth method was used to determine the damping ratio of the system. Figure 9 shows one typical self-power spectral density of acceleration time history on the cap beam in X direction after the bridge model was tested under ground motion with PGA equal to 0.5g. According to this figure, the 1st, 2nd, and 3rd mode fundamental frequency (FF) was determined as 2.44Hz, 17.66 Hz, and 24.98 Hz. The damping ratio (DR) of column at the 1st mode was determined as 11.5%.

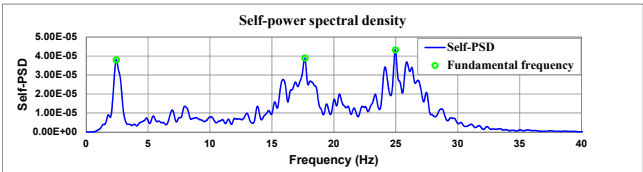


Figure 9. Self- power spectral density of acceleration time history on cap beam.

Figure 10 shows the average 1st mode fundamental frequency and damping ratio of column in both Phase I and Phase II. It was found that, as the tests proceeded, the 1st mode fundamental frequency always decreased, which indicates the accumulated damage in the bridge model. The damping ratio in longitudinal direction increased by 50%, on average, in Phase I. However, the damping ratio in the transverse direction did not change much in both Phase I and Phase II, even the concrete cracked and crushed at the column toe.

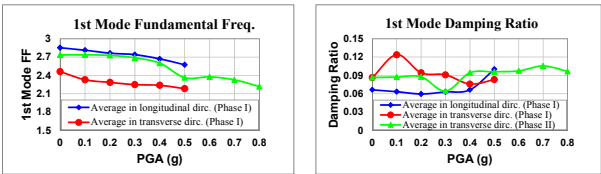


Figure 10. First mode fundamental frequency and damping ratio.

CONCLUSIONS

Several conclusions can be drawn from this research:

- 1) The proposed precast unbonded post-tensioned segmental concrete column can sustain the seismic load under selected ground motions at PGA equal to 0.5g when a

non-seismic rubber bearing is used and at PGA equal to 0.8g when fixed support is used.

2) Concrete crack and crush are mainly found at the column toe, especially in the weak axis (transverse direction). However, no rebar yielding was detected at the plastic hinge, which means that the transverse reinforcement can still provide efficient confinement to the concrete core at the end of the tests.

3) During the tests, the maximum drift ratio at the cap beam was 3.4%, and the maximum joint opening was 9.1mm. No residual displacement was found, and all joint openings were closed at the end of tests.

4) The test was terminated due to the displacement of the superstructure exceeding the measurement range.

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Comparison on Force-based and Displacement-based Approaches for Evaluation of Bridge Reliability under Multiple Hazards

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ABSTRACT

The Strength I Limit State of American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) specification deals with the failure probabilities of the combination of non-extreme loads. This design limit state equation (DLSE) has been fully calibrated for dead load and live load by using the reliability-based approach in the previous research. However, most of other DLSEs including the extreme event I and II, with certain probability-based concepts considered, have not yet been fully calibrated by using the reliability-based approach. This paper presents an evaluation of the bridge reliabilities under the limit state of Extreme Event I combination of dead load, live load and earthquake load, by using a force-based and a displacement-based approach on a set of nine simplified bridge models. Results show that the displacement-based approach comes up with more convergent and accurate reliabilities for selected models.

INTRODUCTION

Designing Bridges to resist extreme hazard loads has been a major concern of American Association of State Highway and Transportation Officials (AASHTO) and the bridge engineering community for decades. In recent decades, a considerable amount of efforts have been devoted to earthquake and wind effects and also extended to other hazards such as scour, storm surge, vessel and vehicular collisions, etc. Analysis and design for these hazard loads have been developed individually because each requires different expertise and knowledge to formulate the “demand” or “capacity” for bridge design. However, a bridge may experience these multiple types of hazards at the same time or in a certain sequence. The bridge capacity needs to be carefully designed by considering its resistance to these multiple extreme hazards in a fair platform to ensure the safety of the bridge structure and the transportation system.

The establishment and implementation of AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specification (AASHTO 2012) was considered

as a milestone in the historical development of bridge design specification. In AASHTO LRFD, both loads and resistance were treated as random variables to take the uncertainties introduced by various external or internal factors into account. For the first time the safety of bridge was evaluated in a quantitative approach using the bridge reliabilities or the risks against various loads and/or load combinations.

AASHTO LRFD can be categorized to four types of design limit states: Strength, Service, Extreme Events and Fatigue. The calibration of AASHTO LRFD consisted of trying out various combinations of load and resistance factors on a number of bridges and their components (AASHTO 2012). Currently the strength design limit state in the AASHTO LRFD is formulated and fully calibrated using the reliability-based approach by Nowak (Nowak 1999) and Kulicki et al. (Kulicki 2007). The extreme event design limit states, however, are constructed by combining the non-extreme load effects with the independently established extreme hazard load effects through professional judgment. Its margin of safety and adequacy has not been assessed quantitatively.

Lee et al. (Lee 2013) presents several different approaches and theoretical backgrounds for the establishment of design limit state equations (DLSE) with a comprehensive consideration of multiple hazards based on proper reliability measures. A particular force-based approach was presented to combine two time-varying loads with the permanent load. In this paper, with limited data/resources, the application of this approach and a displacement-based approach are given to demonstrate the procedure for calibrate the bridge reliability under dead load, live load and earthquake. Furthermore, the results are compared to address the needs for obtaining a more accurate result from the standpoint of bridge engineering community.

GENERAL STEPS FOR EVALUATION OF THE BRIDGE RELIABILITY

Based on the previous research by Nowak (Nowak 1999), the entire procedure to establish the current AASHTO LRFD limit states consists of six steps. A flow chart to demonstrate the general procedure to determine bridge reliability is given in Figure 1. The first four steps can be used to evaluate the bridge reliability, as following:

- 1) Selection of representative bridges. Bridges vary with many parameters, such as structural system, material, span, width, height, skew, etc. To obtain the reasonable result, representative bridges need to be selected to conduct further analyses. Since the considered loads are dead load, live load and earthquake, and the earthquake load effect is mainly applied on the substructure of a bridge, the research focus in this study is placed on the bridge columns with the assumption that other bridge components can be simplified.
- 2) Establishing the statistical data base for load and resistance parameters. Load distributions can be obtained from recorded data and statistical analysis, which commonly reflect the probability of occurrence and related intensity for each hazard. Many researchers proposed reasonable assumptions for simplicity. These simplified assumptions can be used for calculation before more accurate data is obtained.
- 3) Development of load and resistance models. For each given bridge structure, we can calculate three items: the load effect distribution, the nominal value of load effect (Q_1 , Q_2 , Q_3 ...) and the resistance distribution. Note that the load effect

distribution is different from the load distribution, and the failure probability can only be calculated through load effect distribution. For example, the load distribution for earthquake commonly referred to is the distribution of peak ground acceleration (PGA), but the failure probability must be calculated by using the internal force such as bending moment applied on the column.

4) Development of the reliability analysis procedure. Ferry-Borges and Castanheta (Borges and Castanheta 1972), Wen (Wen 1977), Turkstra and Madsen (Turkstra and Madsen 1980), Ghosn et al. (Ghosn 2003), Liang and Lee (Liang and Lee 2012) et al. proposed different methods to calculate the failure probability of a structure using the load effect and structural information provided in steps 2 and 3. This paper focuses on the exhaustive approach proposed by Lee et al. (Lee 2013), which consists of calculation of partial and total failure probabilities and normalization (i.e., converting the load effect distribution into a normal distribution). Lee et al. (Lee 2013) proposed that, for two different time-variant loads, the failure probability of a structure is the sum of three partial failure probabilities from three mutually exclusive events. In this case, the total failure probability is:

$$p_f = p_f^{EQ} + p_f^{EL} + p_f^{LL} \tag{1}$$

where p_f^{EQ} , p_f^{EL} and p_f^{LL} are the partial failure probabilities due to the dead load and earthquake load effect, the combined effect of dead load, earthquake and live load, and dead load and live load effect only, respectively.

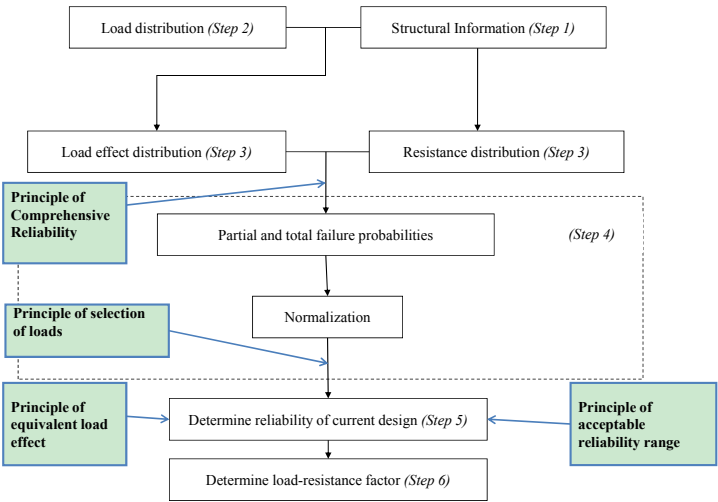


Figure 1 Flow Chart for Determination of the Load Factor

FORCE-BASED APPROACH AND DISPLACEMENT APPROACH TO EVALUATE THE BRIDGE RELIABILITY UNDER EARTHQUAKE

Force-based Approach

Detailed formulations and calculations can be referred to Lee et al. (Lee 2013). Since earthquake is the major cause for the failure of bridge substructure, this section focuses on introducing a force-based approach and a displacement-based approach to evaluate the bridge reliability under earthquake. To obtain an accurate distribution of earthquake load effects, a time-history analysis is used in this study. Several steps are as following:

1) To obtain the peak ground motion (PGA) distribution for one event. According to the definition of Poisson process assumption and introduction in the later section, the distribution of earthquake PGA for one event can be obtained by

$${}_{EV}F_{PGA}(x) = {}_{an}F_{PGA}(x)^{\frac{1}{n_{EQ}}} \quad (2)$$

where ${}_{EV}F_{PGA}(x)$ and ${}_{an}F_{PGA}(x)$ are the cumulative distribution function (CDF) of PGA for earthquakes for one event and the annual CDF, respectively. n_{EQ} is the average number of earthquake occurring in one year. Note that the threshold PGA values for defining an earthquake is an important factor to determine n_{EQ} . Apparently the number of earthquake will increase with the decrease of the threshold value.

2) Calculate the probability density function (PDF) for one earthquake based on their definitions by:

$${}_{EV}f_{PGA}(x) = \frac{d}{dx} {}_{EV}F_{PGA}(x) \quad (3)$$

where ${}_{EV}f_{PGA}(x)$ is the PDF of PGA for earthquakes for one event.

3) Discretize the PDF for one event with a specified interval. In this study, the PGA is divided into 251 levels with an interval of 0.01g (i.e., 0, 0.01g, 0.02g, 0.03g, ..., 2.50g).

4) Select the earthquake ground motions and match them with an objective response spectrum. A total of earthquake ground motions are selected from PEER Next Generation Attenuation (NGA) database. With the objective to develop new ground-motion prediction relations through a comprehensive and highly interactive research program, PEER NGA provides robust and reliable estimates of ground motion than the previous generation of attenuation models developed in 1990s and early 2000s. A total of 99 earthquake records were used. To obtain a more accurate result, the selected ground motions are matched to an objective response spectrum provided by USGS for the selected location. Scale the selected set of earthquake ground motion records to each level of PGA (i.e., 0.01g, 0.02g, 0.03g, ..., 2.50g). The ground motions are linearly scaled by the PGA amplitude in this study.

5) Convert the entire bridge models to a simplified model consists of single or multiple columns. In general, bridge substructures are more vulnerable to earthquake ground motions than superstructures. As mentioned earlier, this research is focused on the bending moment at the column base. To conduct the time-history analysis on a full bridge model is neither efficient nor necessary. Therefore, an equivalent spring to substitute the superstructure is developed to consider the inertial force resisted by the

superstructure.

6) Determine the column property. In this study, the column section is 4 ft in diameter and has 38 #11 bars for longitudinal steel, which amounts to a steel-concrete area ratio of about 1%. In the hinge zones, the columns have confinement steel consisting of #6 spiral bars with a 3.5 in spacing. The concrete and steel used in the columns are 4000 psi and A706 of Grade 60, respectively. The moment-curvature curve of the column is depicted in Figure 2. Note that the axial load of the column has been considered in the moment-curvature curve.

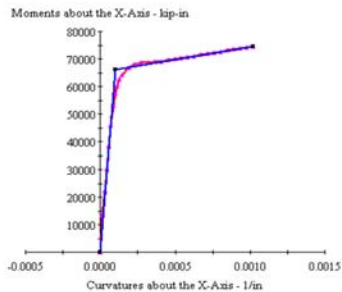


Figure 2 Moment-Curvature Curve and Idealized Bi-Linear Hysteretic Model for the Column

7) Analyze the time-history response of the column by each scaled ground motion, and then record the maximum load responses in the abscissa of the PMF, the corresponding probability of PGA is recorded for ordinate value as depicted in Figure 3.

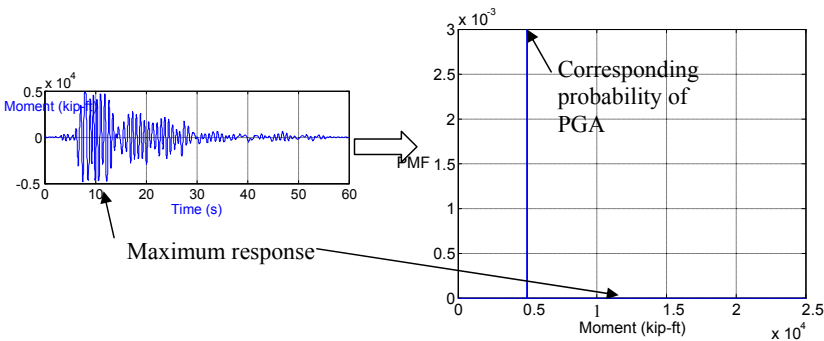


Figure 3 Force-Based Time-History Response of the Column

8) Normalize the PMF to ensure the sum of probability equals 1. As a result, the PMF of earthquake load effect for one event can be obtained as depicted in Figure 4a.

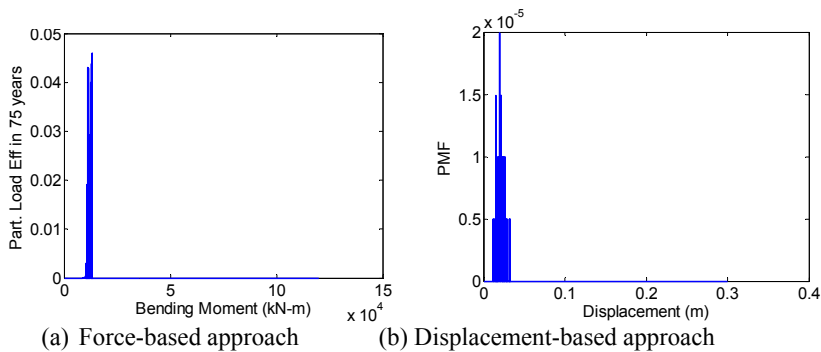


Figure 4 Results of Distribution of Partial Load Effect

Displacement-based Approach

In general, the displacement-based approach follows the same steps as force-based approach. However, in the seventh step, the time-history responses of the column are selected as the displacement instead of bending moment. Figure 5 shows the procedure and Figure 4b shows the result.

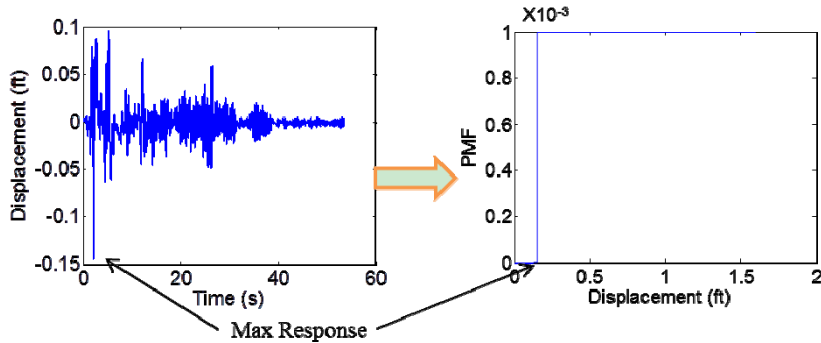


Figure 5 Displacement-Based Time-History Response of the Column

COMPARISON OF TWO APPROACHES

A set of nine simplified bridge models presented in a previous study by Lee et al. (Lee 2013) has been used for the comparison of two approaches. The main span length of the bridges varies from 70 ft to 200 ft so they are in the category of short-to-mid span typical bridges. The reliabilities of these bridges under dead load, live load, and earthquake are calculated and the results are shown in Figure 6. The reliability obtained by the force-based approach varies from 1.27 to 11.54 and that by the displacement-based approach varies from 1.74 to 4.51. It can be concluded that the displacement-based approach provides less variations and thus comes up with more convergent and accurate reliabilities for these selected models.

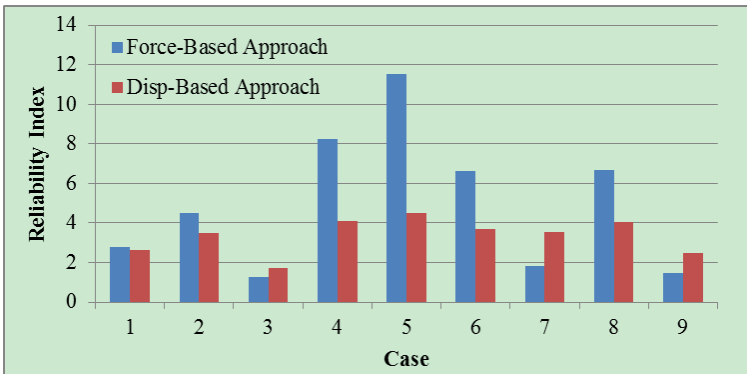


Figure 6 Comparison of Two Approaches

CONCLUSIONS

This paper presents two approaches for the evaluation of the bridge reliabilities under the limit state of Extreme Event I combination of dead load, live load and earthquake load. The difference is on the step of converting load effect to load effect distribution (the seventh step). A set of nine simplified bridge models presented in a previous study by Lee et al. (Lee 2013) has been used for the comparison of two approaches. Results show that the displacement-based approach comes up with more convergent and accurate reliabilities for selected models. Note that the data and sample of bridge models are limited and further calibration are necessary to validate the accuracy of both approaches.

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Anti-Overturning Stability Study of a Continuous Steel-Concrete Composite Girder Bridge with Single Column Piers

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ABSTRACT

Taking an example of a continuous steel-concrete composite beam bridge with single column piers under a ramp, a space finite element model was built. According to the structure characteristics and relevant design manual, an anti-overturning stability analysis of the bridge under the design limit load was conducted, and a practical vehicle load and possible arrangement of different forms of the load were analyzed. A further study on the influence of related parameters of anti-overturning stability was developed to determine the probability of overturning under extreme load arrangement. Finally, a field-loading test with the designed vehicle arrangement was conducted to investigate the experimental anti-overturning stability properties of the designed bridge. The test results were used to further verify the consistency of the related standards about the anti-overturning stability of this kind of bridge, and then some suggestions to avoid the corresponding accidents are provided.

INTRODUCTION

With the development of highways and flyovers, bridge structure has undergone great changes (Liu 2012). To obtain a broader driving view (Li 2012) and to better adapt to the tough terrain and reduce the engineering work of their substructure (Liang 2009), continuous-beam bridges with a single column pier (Ding 2013) are widely accepted. A single column pier can only support a beam with a single pivot. The anti-overturning stability of the structures can be affected under eccentric loads (Zhang 2013). In recent years, overweight vehicles have caused many overturning collapse accidents on this kind of bridge. Currently, bridge designers pay much attention to bending and shearing capacity (Li 2012). Security surplus is retained while overloading (Yao 2013). There are many security risks, such as beam displacement, twisting, bearing disengaging, uneven stressing, etc. (Jiang 2013). An example of a continuous steel-concrete composite beam bridge with a single column pier under a ramp in Beijing was proposed. According to the relevant provisions of “Code for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts” (JTG D62-2012,

Approval Draft) (abbreviation for approval draft later) for a combining loading test, its anti-overturning stability was analyzed. This will provide a reference of overturning design for this stage.

ENGINEERING SITUATION

The subject bridge is the second section of one ramp of a flyover in Beijing. The superstructure is made of three-span continuous steel-concrete composite beams. Both side piers are single-column with double pot bearings, and the bearings can move unidirectionally. Both middle piers are single-column with single pot bearings and the bearings cannot move. The foundation is a pile cap. The load rating for the bridge is highway-I. The layout of the bridge is shown in Figure 1.

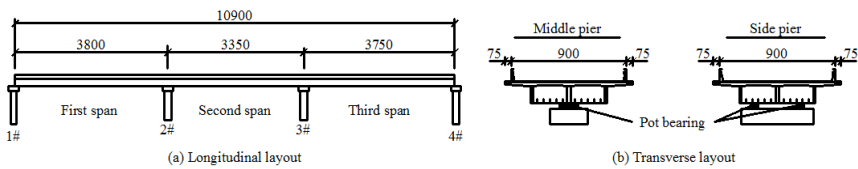


Figure. 1. Layout of bridge (cm).

FINITE ELEMENT MODELING

In recent years, the horizontal alignment of bridge accidents is mostly relatively straight. They are usually located in a straight segment, transition curve, or circular curve of larger curvature radius (Weng 2012). This bridge is located in the initial segment of a transition curve, and the alignment is close to a straight line.

The space finite element model was built by Midas Civil in 2012. Boundary conditions were arranged on the contact surface of the steel box and the bearing. The bearing parameters were input according to real conditions. The full-bridge model has a total of 9,324 nodes and 10,450 units. The finite element model of the full bridge is shown in Figure 2.

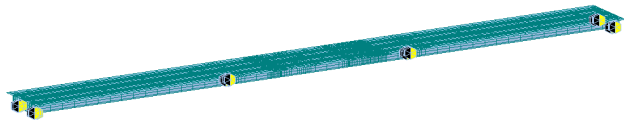


Figure 2. Finite element model of full bridge.

The bridge deck pavement was simulated by a uniform load of 2.8 kN/m². Crash barriers were simulated by a uniform load of 14.5 kN/m². A live load was arranged by the driveway surface. The system temperature and temperature gradient were assumed by related specifications.

ANTI-OVERTURNING STABILITY ANALYSIS

Designed load analysis. According to Section 4.1.10 of the approval draft, the

anti-overturning stability of the bridge was analyzed. The anti-overturning stability factor k_{qf} of the superstructure should meet the following requirements:

$$k_{qf} = \frac{s_{bk}}{s_{sk}} \geq 2.5 \tag{1}$$

Where s_{sk} is the effect of car loading standard value of the superstructure overturning, and s_{bk} is the effect of the standard combination of the superstructure stability. The overturning axis is located in the same side bearing-connection of the bridge centerline, as shown in Figure 3.

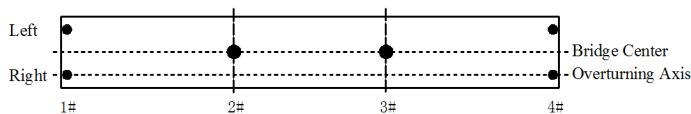


Figure 3. Diagram of overturning axis.

The anti-overturning stability factor k_{qf} of the box-girder bridge is:

$$k_{qf} = \frac{\sum R_{Gi} x_i}{(1 + \mu)(q_k l + P_k) e} \tag{2}$$

Where q_k is the uniformly distributed load of lane load, P_k is the concentrated load of lane load, l is the bridge length, e is the vertical distance of horizontal most disadvantaged lane position to the overturning axis, u is the impact factor, R_{Gi} is the bearing reaction of the finished bridge state, and x_i is the vertical distance of each bearing to overturning axis.

When the bridge state was finished, the stable effect of the superstructure was dead load, such as weight and secondary load. Under dead load, the bearing reaction is shown in Table 1.

Table 1. Bearing Reaction under Dead Load (kN).

Pier Number	1#		2#	3#	4#	
Bearing location	Left	Right	Left	Right	Left	Right
Bearing reaction	1516	1513	5851	5790	1505	1497

Through calculation, $q_k = 10.5kN/m^2$, $P_k = 312kN$, $l = 109m$, $e = 1.99m$ and $u = 0.215$. The anti-overturning stability factor is $12.2 > 2.5$, meeting regulatory requirements.

Unidirectional pressure bearings should not be in the void under the serviceability limit state. The serviceability limit state is weight + secondary load + car loading + system temperature + temperature gradients. The bearing reactions are shown in Table 2.

Overall, under the designed limit loads, the left bearings appeared to have small pull, but the bearings overall are in a state of compression. Therefore, the anti-overturning stability of the bridge meets the desired specification.

Table 2. Bearing Reaction under Serviceability Limit State (kN).

Pier Number	Bearing Location	Maximum reaction		Minimum reaction		Meets specification?
		Resultant	Pull	Resultant	Pull	
1#	Left	1589	0	731	-21	Y
	Right	2809	0	1428	0	Y
2#	Middle	7916	0	5188	0	Y
3#	Middle	7851	0	5119	0	Y
4#	Left	1580	0	719	-12	Y
	Right	2789	0	1409	0	Y

Note: Positive represents pressure and negative represent spull.

Stability factor analysis. From formulas (1) and (2), it can be seen that the anti-overturning stability factor is the torque ratio of dead load s_{bk} and live load s_{sk} in fact. S_{bk} is influenced only by reacting force R and the vertical distance x . If the superstructure is determined, the bearing reaction is certain theoretically and is the biggest affecting factor is the bearing spacing. In the same live load, with the larger bearing spacing, the s_{bk} value can be larger, the e value can be smaller, and k_{qf} can be larger. Therefore, the whole structure is more stable.

For s_{sk} , the maximum impact parameter is the value of the live load. This study focuses on s_{sk} against the overturning stability factor. The real safety factor of a steel-concrete composite beam bridge is validated for formula (1) and (2). A further extreme loading arrangement that may happen to capsize is identified.

Effect of multiple designed loads. *Condition 1:* under the serviceability limit state, considering two times the highway-I. *Condition 2:* under the serviceability limit state, considering three times the highway-I. Under two and three times the highway-I lane loads, the anti-overturning stability factors were 6.1 and 4.1, respectively, greater than 2.5 and meeting the requirements of the formula. From Section 4.1, it shows that bearing negative reaction appears in the minimum data. The bearing minimum reactions of conditions 1 and 2 are shown in Figure 4.

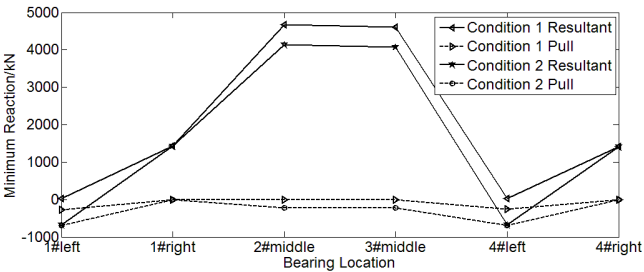


Figure 4. Bearing minimum reactions of conditions 1 and 2 (kN).

From Figure 4, under three times highway-I lane loads, the left bearings of the side piers are both in a state of tension, and the bearings of the middle pier also begin to appear to greatly pull. It can be assumed that bearings have been in void and a capsizing accident has occurred.

Under two times highway-I lane loads, while the bearing result does not appear to pull, the left bearings of the side piers partly appear to greatly pull and the bearing tension of the middle pier begins to emerge. The corresponding medial and lateral bearings have a large uneven reaction on the side piers. In fact, the box girder under such conditions is subject to large bending and torsion, and great flexural and torsional deformation may occur. Excessive angle deformation leads to sliding bearings damage. Therefore, this condition may cause the overturning.

In summary, the anti-overturning stability factor of the approval draft is small for steel-concrete composite beams. When designing, simply relying on the anti-overturning stability factor is insufficient. A further check of bearing disengaging is needed to ensure safety.

Effect of actual vehicle load. A 55t vehicle was simulated by the common three-axis. Vehicles were densely arranged on the side of the bridge according to the actual position. A schematic diagram of the vehicle is shown in Figure 5.

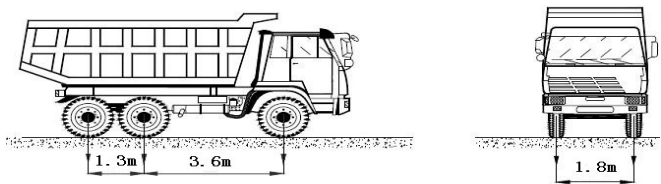
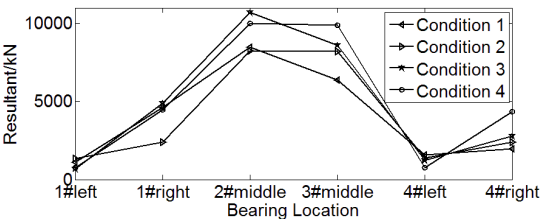


Figure 5. Schematic diagram of vehicle.

- The actual layout of the vehicle load was divided into four kinds of conditions:
- Condition 1: under the serviceability limit state, the vehicles act on the right of the first span.
 - Condition 2: under the serviceability limit state, the vehicles act on the right of the second span.
 - Condition 3: under the serviceability limit state, the vehicles act on the right of the first and second spans.
 - Condition 4: under the serviceability limit state, the vehicles act on the right of the full bridge.

The bearing reactions of Conditions 1–4 are shown in Figure 6.



(a) Bearing resultant

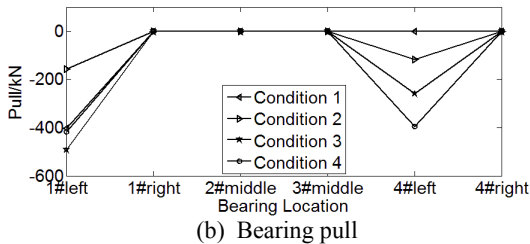


Figure 6. Bearing reactions of Conditions 1–4 (kN).

Condition 1 shows that the left bearing appear to have a large pull only on the 1# pier. Each bearing is overall in a state of compression. Therefore, when the vehicles act on the right of the first span, the probability of a capsizing accident is small.

Condition 2 shows that each bearing is in a state of compression. But the left bearings appear to have a large pull on the 1# and 4# piers. Therefore, when the vehicles act on the right of the second span, the probability of a capsizing accident is larger than in Condition 1.

In Condition 3, although each bearing is overall in a state of compression, the resultant reduced and pull increased significantly on the left bearings. Corresponding medial and lateral bearings appear to have a larger imbalanced reaction. Therefore, when the vehicles act on the right of the first and second spans, the probability of a capsizing accident is larger than in Condition 2.

From Condition 4, contrasting with Condition 3, each bearing reaction is basically the same. Therefore, when the vehicle load acts on the right of the full bridge, the probability of a capsizing accident is almost identical to Condition 3.

From the above, when a continuous steel-concrete composite beam bridge with a single column pier is designed for higher probability Conditions 1 and 2, the structural strength and normal use need to be full evaluated. For lesser probability Conditions 3 and 4, only the evaluation of safety indicator is necessary. For normal operational phase, heavy vehicles filling the same side of the two spans should be avoided.

Field loading test.

Loading scenario. Based on the above calculations, the field loading tests were done to verify the anti-overturning stability of the bridge according to the original designed load levels. The testing subjects of the ramp were the first and second spans. The layout of the measured points was the same on both spans. The layout of the test section is shown in Figure 7.

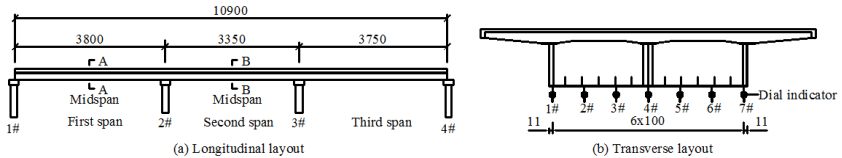


Figure 7. Layout of test section (cm).

According to the mechanical characteristics and theoretical calculations of a continuous steel-concrete composite beam with a single column pier, the anti-overturning test scenario of the ramp shows as follows:

Condition 1: test the maximum torque of the midspan section of the first span under the most disadvantaged live loads, testing the load using eccentric layout.

Condition 2: test the maximum torque of the midspan section of the second span under the most disadvantaged live loads, testing the load using eccentric layout.

Test results analysis. Under the most disadvantaged eccentric loading of Conditions 1 and 2, deflection results of the midspan section of the first span are shown in Table 3 and Figure 8.

Table 3. Measured and Calculated Deflection (mm).

Point Number	A-A Section			B-B Section		
	Calculation	Measuring	Factor	Calculation	Measuring	Factor
1#	-6.47	-6.19	0.957	-4.91	-4.3	0.876
2#	-6.11	-5.56	0.911	-4.38	-4.1	0.937
3#	-5.50	-4.83	0.878	-3.51	-3.2	0.912
4#	-5.10	-4.51	0.885	-2.93	-2.4	0.820
5#	-4.73	-4.16	0.879	-2.38	-2.1	0.884
6#	-4.19	-3.84	0.917	-1.53	-1.2	0.783
7#	-3.85	-3.11	0.807	-1.01	-0.8	0.790

Note: Positive represents upward and negative represents downward.

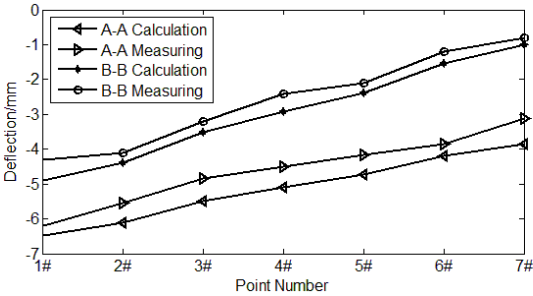


Figure 8. Measured and calculated deflection relationship (mm).

Under the most adverse torque, the check factor of the measured deflection of the midspan section of both spans ranged between 0.783–0.957. Each measured deflection is greater than the corresponding calculated values. The vertical rigidity of the bridge structure can meet the designed load requirement. The measured deflections are downward in the midspan section of both spans, which reflects a healthy overturning resistance of the box girder.

The results show that the anti-overturning stability of the ramp meets the requirement of the codes. The maximum test check factors are close to 1, validating that the anti-overturning factor of the approval draft is smaller. For this kind of bridge, when operating, it is recommended to establish a temporary vehicle monitoring system

to master the traffic loads timely and accurately. The entrance of vehicles within 50cm of the curb should be restricted as much as possible. Overload vehicles should be strictly prohibited to pass the bridge. In addition, the ramp bridge traffic flow should be ensured to avoid heavy vehicles crossing bridge at the same time.

CONCLUSIONS

Research conclusions are summarized as follows:

- (1) The measured and theoretical displacement of field loading tests has the same trends. The values are in good agreement. This reflects that the load model and the boundary conditions of the finite element model are in accordance with the actual.
- (2) For a box girder of the side pier setting double bearings, increasing the spacing of the bearings helps to improve the anti-overturning stability.
- (3) For continuous steel-concrete composite beams, the side bearings appear to pull under the serviceability limit state. Therefore, tensile measures of bearings need to be considered.
- (4) For continuous steel-concrete composite beams, the anti-overturning factor of the approval draft is smaller. In design, checking the bearing in a void is needed further.
- (5) Such bridges should have warning signs. Overload vehicles should be strictly prohibited to pass the bridge. Traffic flow should be ensured on the ramp. Heavy vehicles should be restricted from passing the bridge at the same time.

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Application of Multi-Scale Finite Element Modeling in Bridge Seismic Damage and Collapse Analysis

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ABSTRACT

To improve the calculation efficiency and accuracy of numerical simulation of structure seismic damage, multi-scale finite element modeling is introduced in this paper. Two different treatment schemes of interface coupling between different-scale models by using a force equilibrium condition and a deformation compatibility condition were researched. Finally, a bridge that collapsed in the Wenchuan earthquake is used as an example to illustrate the suggested approach in this paper. A multi-scale finite element model of a bridge was established based on ABAQUS software. The seismic damage and the collapse process were simulated and compared with the actual seismic damage investigation to verify the validity of multi-scale finite element modeling. The research indicates that multi-scale finite element model can be applied to reasonably and effectively simulate the local damage and whole collapse of a complex bridge due to an earthquake, and the treatment scheme by using a force equilibrium condition can meet the stress and deformation compatibility well at the interface between different-scale models.

INTRODUCTION

At present, two kinds of methods are applied in engineering nonlinear numerical simulation: (1) nonlinear calculation of the whole structure based on macroscopic model with member or shell element; (2) nonlinear calculation of complex components and local areas of structure based on precise microscopic model with solid element. The nonlinear analysis of the whole structure based on a macroscopic model cannot effectively reflect the damage development process and the failure mechanism of local components and areas of structure (Sun, Li and Chen 2007). Although, the nonlinear analysis based on a precise microscopic model can simulate the damage process and failure mechanism properly, it is impractical to analyze the whole nonlinear behavior of a complex structure due to the limitation of numerical calculation capacity. In addition, it is more difficult to determine the boundary conditions properly while a substructure method is applied, especially for nonlinear time history analysis of a structure subjected to complex dynamic loads.

Therefore, to satisfy the actual requirements of engineering calculation, it is necessary to develop an appropriate numerical simulation method to accurately

reflect the damage process and failure mechanism of a structure with an acceptable computational complexity. The multi-scale finite element numerical simulation method is an effective approach to solve this problem (Ladeveze, Nouy and Loiseau 2002; Khandelwal and El-Tawil 2005). In general, the multi-scale modeling method includes scale separation and scale coupling. The former focuses on establishing different-scale models in different parts of a structure, and the latter puts emphasis on the interface connection treatment between the macroscopic model and the microscopic model (Lin, Lu and Ye 2010; Broughton, et al. 1999).

According to the ideas mentioned above, a multi-scale finite element modeling method is introduced in this paper, and two different schemes for dealing with the coupling at the interface between different-scale models by a force equilibrium condition and a deformation compatibility condition are researched. To illustrate the suggested approach in this paper, a bridge that collapsed in the Wenchuan earthquake is used as an example, and the precise microscopic finite element model is established based on ABAQUS software. The seismic damage and progressive collapse process are simulated.

TREATMENT SCHEME OF INTERFACE CONNECTION BETWEEN DIFFERENT-SCALE MODELS

The treatment scheme of an interface connection is a key problem in the multi-scale numerical simulation of a structure. The stress distribution rationality and deformation compatibility should be satisfied simultaneously in the interface between different-scale models. In general, the interface connection includes a beam-shell element connection, a beam-solid element connection, and a shell-solid element connection. Their interface connection principles are the same. There are two major schemes applied in dealing with interface connections between different-scale models: (1) the force equilibrium method and (2) the deformation compatibility method. In this paper, the interface connection between a beam element and solid elements is given as an example to analyze the theory and applicability of two treatment schemes of interface connections.

In three-dimensional space, a solid element node has only three translational degrees of freedom but no rotational degree of freedom. However, a beam element node has six degrees of freedom. The displacement relationships of node at the interface between the solid element and the beam element subjected to bending moment and axial force are shown in Figure 1. The deformation compatibility of the element node in a multi-scale model satisfies the plane section assumption. Therefore, the interface is regarded as a rigid body under external load, and the node of beam element is regarded as a reference point in the rigid body. The translational displacement of a solid element node is compatible with the displacement of a corresponding node in a rigid body.

Let Δx_{si} and Δz_{si} be the X and Z direction displacements of solid element node S_i respectively. Let Δx_B , Δz_B and α be the X, Z direction displacement and Y direction rotation angle of beam element node B respectively. Obviously, the displacement of the beam element node is coupled with that of the solid element node.

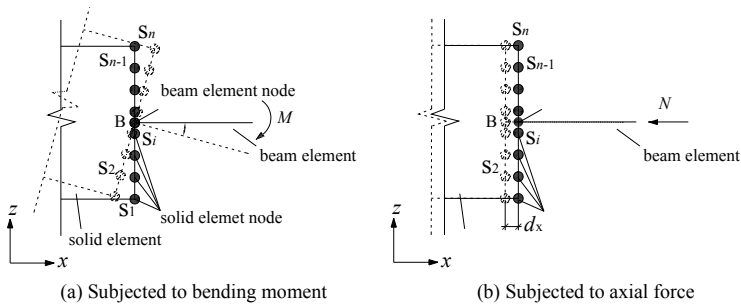


Figure 1. Node displacement relationship in deformation compatibility condition.

When a beam element deforms actively, the displacement of solid element node S_i can be determined according to the displacement of beam element node B. The displacement of solid element node S_i is expressed as follows.

$$\Delta x_{si} = \Delta x_B + r_{si} \cdot \sin \alpha ; \quad \Delta z_{si} = \Delta z_B + r_{si} (\cos \alpha - 1) \tag{1}$$

When a solid element deforms actively, the displacement of beam element node B can be determined according to the displacements of all solid element nodes in the interface. The displacement of beam element node B is expressed as follows.

$$\tan \alpha = (\Delta x_{sn} - \Delta x_{s1}) / h ; \quad \Delta z_B = (\sum_{i=1}^n \Delta z_{si}) / n ; \quad \Delta x_B = (\sum_{i=1}^n \Delta x_{si}) / n \tag{2}$$

Where $r_{si} = z_{si} - z_B$, z_{si} , and z_B are Z-axis coordinate values of solid element and beam element, and h is Z-direction section height of solid element.

To illustrate the numerical simulation effect of a treatment scheme with a deformation compatibility condition, a cantilever column is given as an example. The column is fixed at the bottom and applied with an axial force and a shear force on the top. Different models including a solid model, multi-scale models, and a beam model are established, as shown in Figure 2 based on ABAQUS software. The analysis results are shown in Figure 3.

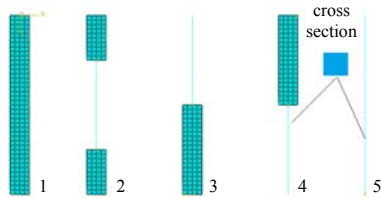


Figure 2. Finite element models of a cantilever column.

It can be seen from Figure 3(a) that when shear force is applied, the deformation and stress results of the 1th–5th models are in agreement with each other. The results also show that deformation compatibility of element connection in interface is obtained, and there is no stress concentration phenomenon. However, Figure 3(b) shows that when axial force is applied, the stress results of the multi-scale models are obviously inconsistent with those of single-scale

models. The stress concentration phenomenon appears both at the interface and the fixed bottom region. The node of solid element in the interface is restrained due to “Poisson effect.”

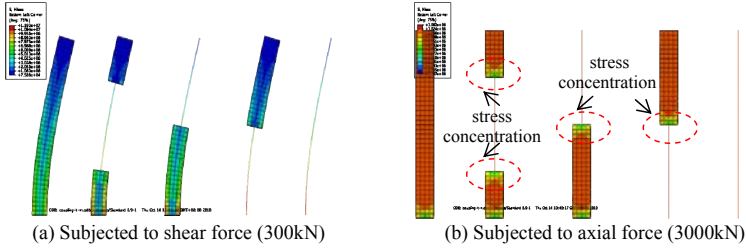


Figure 3. Stress distribution of column in deformation compatibility condition.

Then, the plane section assumption is satisfied when the treatment scheme with a force equilibrium condition is adopted. The relationship of reaction force of node at the interface of solid-beam element under external load is shown as Figure 4.

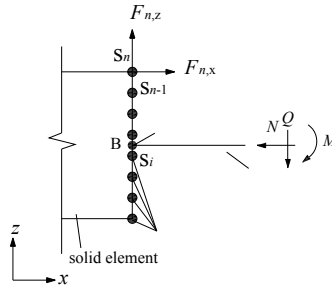


Figure 4. Node reaction force relationship in a force equilibrium condition.

Let $\mathbf{P} = [N, M, Q]^T$ and $\mathbf{F}^i = [F_x^i, F_z^i]^T$ be the internal force of beam element node B and solid element node S_i , respectively. When the beam element deforms actively, the internal force of solid element node S_i is expressed Eq. (3). When the solid element deforms actively, the internal force of beam element node B is expressed as Eq. (4).

$$F_x^i = \frac{A_i}{A} N + \frac{Mr_{si}A_i}{I}; \quad F_z^i = \frac{A_i}{A} Q \quad (3)$$

$$M = \sum_{i=1}^n F_x^i r_{si} \quad N = \sum_{i=1}^n F_x^i \quad Q = \sum_{i=1}^n F_z^i \quad (4)$$

Where A is the cross-section area of solid element, A_i is the influence area of solid element node S_i , and I is the inertia moment of solid element cross section.

To illustrate the numerical simulation effect of treatment scheme with the force equilibrium condition, the analysis of cantilever column model shown in Figure 2 is carried out and the results are shown in Figure 5.

From Figure 5, it can be seen that the results of deformation and stress distribution of the 1th–5th models are in good agreement with each other. The

element connection at the interface also satisfies the deformation compatibility and there is no stress concentration phenomenon.

According to above analysis, for multi-scale modeling of a structure under static load, the treatment scheme with a force equilibrium condition is more accurate than that with a deformation compatibility condition.

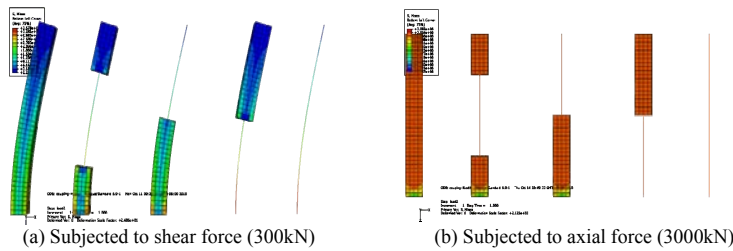


Figure 5. Stress distribution of column in a force equilibrium condition.

TREATMENT SCHEME WITH FORCE EQUILIBRIUM CONDITION IN DYNAMIC AND NONLINEAR ANALYSIS

The accuracy of dynamic properties of a structure established by a multi-scale model based on a force equilibrium condition was verified. The natural frequencies of a structure in Figure 2 were computed, and the results are shown in Table 1.

Table 1. Natural Frequencies of a Structure by Different Models.

	Model 1	Model 2	Model 3	Model 4	Model 5
1st frequency	5.64	5.71	5.83	5.67	5.84
2nd frequency	5.64	5.71	5.83	5.67	5.84
3rd frequency	34.0	34.8	34.56	34.46	35.1

It can be seen from Table 1 that the first and second frequencies are the same because the structure is symmetrical about two horizontal axes. The frequencies of different models are almost same; the frequency of the solid model is the lowest, the frequency of the beam model is the highest, and the frequencies of the multi-scale models are between them. Obviously, the difference of results is mainly attributed to the difference of the natural dynamic properties of the solid element and the beam element.

To verify the accuracy of a treatment scheme with a force equilibrium condition in nonlinear analysis, models were analyzed under monotonic loading. The material stress-strain relationship is defined by the bilinear model. The transverse displacement was applied to the column top, and the load-displacement curve was computed, as shown in Figure 6. It indicates that the results of different models fit well.

In addition, the time history analysis of models due to earthquake ground motion was also carried out, and the results are shown in Figure 7. The displacement responses of the three models fit well in the earlier period; however, some difference of response amplitude exists in the latter period. This is mainly because the solid element and the beam element have different assessing rules of

material entering plastic phase and natural dynamic properties.

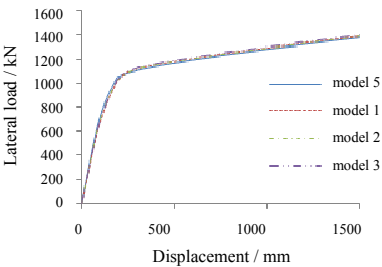


Figure 6. Load-displacement curve of column top.

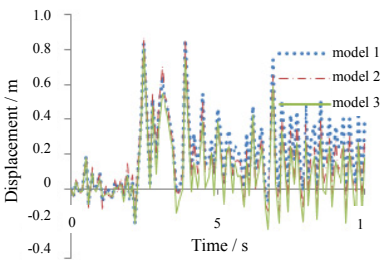


Figure 7. Displacement time history of column top.

NUMERICAL SIMULATION OF EXAMPLE BRIDGE

A curve girder bridge that was severely destroyed in the Wenchuan earthquake is used as an engineering example to illustrate the suggested modeling approach in this paper. A multi-scale finite element model of the bridge was established based on ABAQUS software, and the nonlinear seismic responses and collapse processes were simulated.

First, according to the seismic damage investigation and the numerical simulation result using a solid element model, the main damaged parts were determined, as shown in Figure 8. Therefore, the damaged parts of bridge pier were modeled by solid element and others were modeled by beam element. The multi-scale finite element model of the pier is shown in Figure 9.

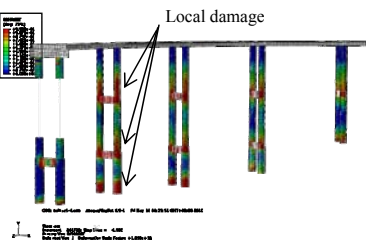


Figure 8. Numerical simulation result by using solid element model.

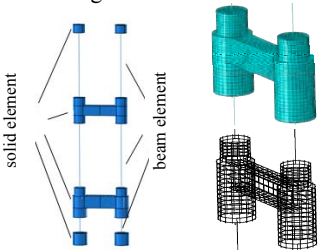


Figure 9. Multi-scale finite element model of piers.

In the model, the material of beam element was assumed to be elastic, and the reinforced concrete of the solid element was simulated by a separate plastic model. The Jeeho Lee concrete model (Lee and Fenves 1998) and the bilinear steel bar model were considered, respectively. The force equilibrium condition was applied to deal with the interface connection between the solid model and the beam model.

The girder model was divided into two parts; the parts on the top of bearing and bracket were modeled by a solid element, and others were modeled by a shell element, as shown in Figure 10. The plastic damage constitutive relation

was adopted in the solid model, and the elastic constitutive relation was adopted in the shell model. The tensile and compressive effects of the steel bar in the girder were simulated by nonlinear spring.

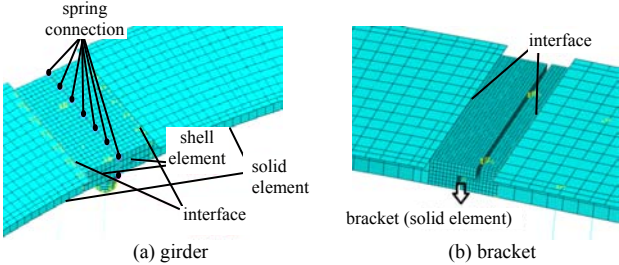


Figure 10. Multi-scale finite element model of girder.

In addition, bearing was simulated by the link element and contact friction element, and the collision effect was simulated by three-dimensional contact friction element through search algorithm. The effects of structure-pier-soil interaction and seismic traveling wave were not considered. The actual seismic record was adopted as the earthquake input of structure in three directions.

The numerical simulation results of seismic damage and collapse are shown in Figure 11 and Figure 12, respectively. It was found that the actual seismic damage process can be simulated accurately using the multi-scale modeling in this paper.

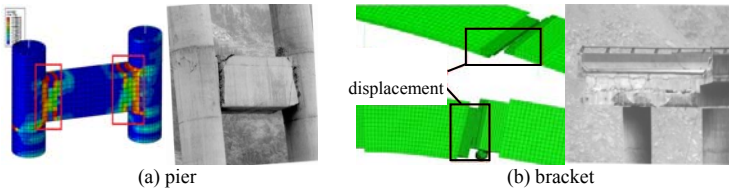


Figure 11. Seismic damage of pier and bracket.

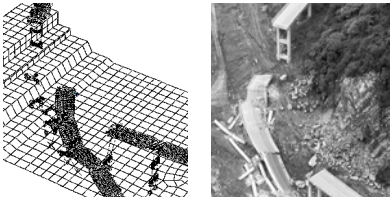


Figure 12. Seismic collapse of bridge.

CONCLUSIONS

(1) When a treatment scheme with a deformation compatibility condition is adopted to deal with the interface coupling of a multi-scale model of a cantilever beam subjected to an axial force, the node of the solid element in the connection interface is restrained due to the “Poisson effect” and a stress

concentration phenomenon appears in the interface connection regions. The results of multi-scale models are obviously inconsistent with the results of the single-scale model.

(2) When a force equilibrium condition is applied to the treatment scheme of the interface connection, the element node at the interface satisfies the deformation compatibility, and the stress concentration phenomenon is eliminated. The results of multi-scale models are in good agreement with the results of the single-scale model.

(3) By establishing the multi-scale finite elements of a bridge and using the treatment scheme with a force equilibrium condition, both the global and local mechanism properties and response behaviors of a structure can be obtained, and the actual seismic damage and collapse process can be simulated accurately with a limited computation resource condition.

ACKNOWLEDGMENT

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Research on the Seismic Performance of Reinforced Concrete Hollow Piers

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ABSTRACT

To study the seismic performance of reinforced concrete hollow piers, models with different axial compression ratios, reinforcement ratios, and height were established and their seismic performance was observed by nonlinear analysis based on the finite element analysis software OpenSees. The results indicate that hollow piers have good seismic performance, and the change of the relative property parameters will affect the performance. The research result of this paper can be used in bridge engineering.

INTRODUCTION

At present, high piers of long-span bridges usually use reinforced concrete hollow piers. Hollow pier have advantages such as higher initial stiffness, good stability, materials-saving, cost-effective, etc., and are used extensively in bridge engineering. The seismic performance of reinforced concrete hollow piers is important for structure engineers. Unfortunately, there are few studies on hollow piers, and the seismic performance of hollow piers still needs study.

Mander (1983) studied reinforced concrete hollow piers with rectangular and circular sections and observed their seismic performance through a quasi-static test. The results prove that the constraints of stirrups can improve the seismic performance of hollow piers, and the neutral axis of position will influence the seismic performance of circular section hollow piers. Song (2005) designed three rectangular section hollow piers whose shear span ratio was 0.8, and quasi-static tests were performed. Results proved that hollow piers have better ductility, and the increasing of wall thickness and reinforcement ratio can improve the piers' stiffness and energy dissipation capacity.

Piers support superstructures and transmit dead load and live load. They are vulnerable components in earthquakes. The seismic performance of piers affects the performance of bridges. The reason most of bridges collapse is the failure of broken piers. Therefore, seismic performance is an important issue in earthquake engineering.

To study the seismic performance of hollow piers, 12 hollow pier models were established through the software OpenSees, and the main factors such as longitudinal steel ratio, axial compression ratio, stirrup ratio, and so on that influence seismic performance of hollow piers were observed by pushover analysis and dynamic history analysis.

NUMERICAL PIER MODEL AND INPUT GROUND MOTION

Numerical pier model. The test models of the Beijing University of Technology (2011) were referred to when establishing the numerical pier model. In total, 12 models were established, and the models were divided into three groups with heights of 1440mm, 2880mm, and 3600mm. The sectional dimension of models are the same, i.e., the overall dimension is 500mm×360mm and the hollow size is 260mm×120mm. Specific parameters are shown in Table 1.

Table 1. Specific Parameters of Models.

Model	Height (mm)	Axial pressure (MN)	Axial compression ratio	Longitudinal reinforcement		Stirrup		
				Diam. (mm)	Reinforce-ment ratio	Diam. (mm)	Spacing (mm)	Volume stirrup ratio
S1	1440	0.28	0.1	8	0.014	6	40	0.035
S2	1440	0.28	0.1	10	0.021	6	40	0.035
S3	2880	0.28	0.1	8	0.014	6	40	0.035
S4	2880	0.28	0.1	10	0.021	6	40	0.035
S5	2880	0.56	0.2	8	0.014	6	40	0.035
S6	2880	0.56	0.2	10	0.021	6	40	0.035
S7	2880	0.28	0.1	10	0.021	6	55	0.025
S8	3600	0.28	0.1	8	0.014	6	40	0.035
S9	3600	0.28	0.1	10	0.021	6	40	0.035
S10	3600	0.56	0.2	8	0.014	6	40	0.035
S11	3600	0.56	0.2	10	0.021	6	40	0.035

Input ground motion. Three ground motion recorders were selected. According to the code for the seismic design of buildings (GB50011-2011, in Chinese), in areas where the seismic fortification intensity is 8, the peak ground acceleration of these recorders should be set as 70 cm/s2 (for frequent earthquake) and 400 cm/s2 (for rare occurrence earthquake). The relative information of the recorders is shown in Table 2.

Table 2. Information of Selected Ground Motion.

Earthquake Name	Date	Step size	PGA / g
Imperial Valley-06	1979-10-15 23:16	0.005	0.3092
Superstition Hills-02	1987-11-24 13:16	0.01	0.4509
EI Centro	1940	0.02	0.3188

ANALYSIS OF SEISMIC PERFORMANCE OF THE PIERS

Analysis of limit load. The limit loads and corresponding displacements of the models are listed in Table 3. As shown in Table 3, the factors that influence seismic performance of reinforced concrete rectangular hollow bridges, such as longitudinal

reinforcement ratio, axial compression ratio, and stirrup reinforcement ratio, can be compared by analyzing the data in Table 1 and Table 2.

Table 3. Limit Load and Corresponding Displacement.

Model	Crack		Surrender		Peak load	
	F _{cr} /KN	U _{cr} /mm	F _v /KN	U _v /mm	F _d /KN	U _d /mm
S1	132.06	2	171.6	11	212.9	70
S2	207.86	5	241.5	15	321.26	139
S3	57.12	5	74.49	16	106.57	282
S4	57.6	4	109.1	24	143.76	245
S5	60.93	4	88	13	123	247
S6	72	5	108	15	148.9	117
S7	56	4	82	12	112.14	47
S8	42.89	6	56.81	21	85.27	441
S9	40.8	5	68.2	19	95.91	89
S10	47.4	6	68.99	18	92.06	194
S11	49.42	6	88.39	25	119.96	199
S12	33.48	4	64.3	18	91.22	89

- (1) With the increase of longitudinal reinforcement ratio, the seismic performance of short piers has obvious improvement, but the effect on the middle-size piers and high pier is not significant.
- (2) The increase of the axial compression ratio can improve the bearing capacity of the piers. When the reinforcement ratio is higher, the increase of the axial compression ratio can improve the bearing capacity of the piers.
- (3) The volume stirrup ratio can improve the bearing capacity of bridge piers, but for the two high-width piers, the improvement is not obvious.

In the end, the data of three typical models, S1, S3, and S8, which have the same longitudinal reinforcement ratio, axial compression ratio, and volume stirrup ratio but different height, were compared. With the increase of height, cracking load, yield load, and maximum load drops, while cracking displacement, yield displacement, and maximum displacement increase step by step. It indicates that as the height increases, the bearing capacity of the bridge piers will reduce gradually and the deformation ability is gradually strengthened.

Ductility analysis. A pushover analysis was performed, and the results are shown in Table 4. It can be seen from Table 4 that, on the whole, the piers’ ductility coefficient is larger than that of high piers and low piers. The factors that influence ductility were observed:

- (1) Influence of longitudinal reinforcement ratio – in the short piers, with the increase of reinforcement ratio, the ductility factor increases, and it is opposite in the low pier and high piers.
- (2) Influence of axial compression ratio – the influence of the axial compression ratio on the ductility factor of short- and middle-size piers has no definite trend.
- (3) Influence of stirrup reinforcement ratio – the decrease of the stirrup

reinforcement ratio has little influence on the bearing capacity of the model, but it influences the ductility coefficient greatly. The influence of the stirrup ratio on the ductility of high piers is less than that of the other piers.

Table 4. Value of Displacement and Ductility Ratio.

Model number	Cracking displacement	Yield displacement	Maximum displacement	Ductility ratio
	U_{cr}/mm	U_y/mm	U_d/mm	μ_s/mm
S1	2	11	70	6.36
S2	5	15	139	9.26
S3	5	16	282	17.6
S4	4	24	245	10.2
S5	4	13	247	19
S6	5	15	117	7.8
S7	4	12	47	3.9
S8	6	21	441	21
S9	5	19	89	4.68
S10	6	18	194	10.78
S11	6	25	199	7.96
S12	4	18	89	4.94

Analysis of dynamic responses. The dynamic responses of the model were obtained by using the OpenSees program, and the top displacement and base shear were used to study the seismic performance of the piers. The relative results are shown in Table 5.

Table 5. Time-History Response Results in Rare Earthquakes.

Model	Vertex maximum displacement (mm)			Basal maximum shear (KN)		
	Imperial Valley-06	Superstition Hills-02	EI Centro	Imperial Valley-06	Superstition Hills-02	EI Centro
S1	0.87	3.3	6.1	102.727	140.885	155.562
S2	0.81	2.6	2.4	103.222	159.475	156.069
S3	33.9	75.4	39.8	82.781	90.674	89.778
S4	19.5	45.1	35.5	101.611	116.205	111.943
S5	159.4	129.8	65.4	118.29	115.82	106.22
S6	77.8	88.4	55.8	139.436	142.427	133.345
S7	18.7	47	38.4	101.066	116.424	117.98
S8	78.4	101.9	54.5	69.395	71.023	67.344
S9	66.5	96.6	59.9	92.569	97.094	91.208
S10	309.6	206.2	67.4	96.745	109.56	81.323
S11	190.5	180.2	72.4	119.51	118.683	106.224
S12	65.5	97	59.9	91.976	96.611	90.757

From Table 5, it can be found that:

- (1) The improvement of the longitudinal reinforcement ratio will reduce the top displacement of the model while improving the base shear.
- (2) The axial compression ratio influences the seismic response greatly. Increasing the axial compression ratio will improve the top displacement, but will have little influence of the base shear.
- (3) Models that have a low axial compression ratio and a high longitudinal reinforcement ratio have good seismic performance.
- (4) The change of the volume stirrup ratio has a little influence on dynamic response.

Energy dissipation capacity analysis. The capacity of energy dissipation of piers can be observed by using the load-displacement hysteresis curve under repeated load. For the most part, the model's energy dissipation capacity is better with the area of the hysteresis curve shape fuller and larger. According to the results of time history analysis, the hysteresis curve of the typical models is shown in Figure 1 and Figure 2.

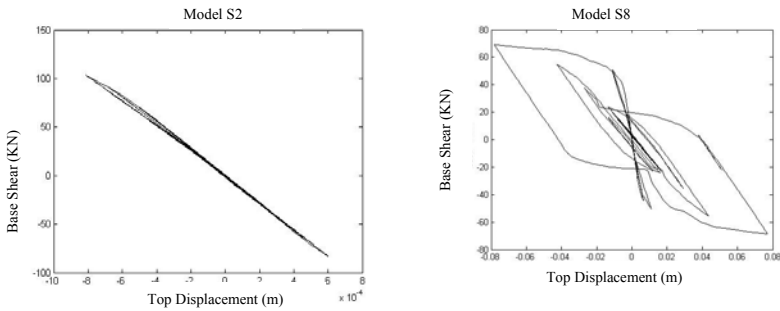


Figure 1. Hysteresis curve of S2 and S8 for Imperial Valley-06 Earthquake.

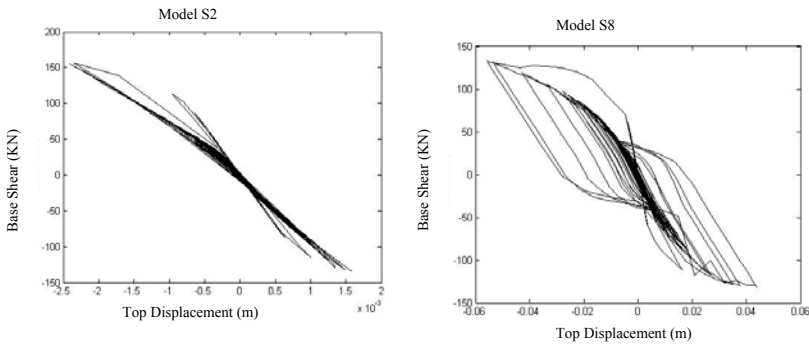


Figure 2. Hysteresis curve of S2 and S8 for EI Centro Earthquake.

As shown in Figure 1 and Figure 2, the energy dissipation capacity of short piers is

very small, and the surrounded area of the hysteresis curve is tiny. That is to say, for short piers, the sliding phenomenon, shear failure, or bending shear failure may occur when an earthquake occurs.

In general, the energy dissipation capacity of short piers is poorer than high piers. With increasing height, the energy dissipation capacity of piers gradually improves. The volume stirrup ratio has a great impact on the energy dissipation capacity. Energy dissipation of S8 and S6 is relatively good in all models.

CONCLUSIONS

To study the seismic performance of hollow piers, 12 reinforced concrete hollow pier models were established, and pushover analysis and dynamic analysis were performed using OpenSees. Factors such as axial compression ratio, reinforcement ratio, height, and stirrup ratio that affect the seismic performance were observed. The following conclusions were obtained:

(1) With the increase of the longitudinal reinforcement ratio, cracking can be significantly delayed for hollow piers. For both high piers and low piers, the increase in longitudinal reinforcement can increase the bearing capacity of the component. The increase of the axial compression ratio can improve the bearing capacity of the piers. Hollow piers with a low axial compression ratio and a high longitudinal reinforcement steel ratio have good seismic performance. The volume stirrup ratio can improve the bearing capacity of piers, but for high piers, the increase is not significant.

(2) Ductility is decided by the longitudinal reinforcement ratio, the axial compression ratio, and the stirrup ratio volume. Ductility ability is the result of several factors, and cannot be decided by only one factor.

(3) The energy dissipation capacity of short piers is poorer than of high piers. With increasing pier height, the energy consumption capacity gradually increases. The volume stirrup ratio has a great impact on the energy dissipation capacity.

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Study on the Seismic Behavior of the Large-span Curved Continuous Rigid Frame Bridge under Different Curvature Radius

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ABSTRACT

With the rapid development of highway projects, more and more long-span curved continuous rigid frame bridges are built. However, there are very few research about the dynamic characteristics of curved continuous rigid frame bridge, especially the seismic response characteristics to the date. To analyze the seismic response, a long span continuous rigid frame curved bridge - WaYaoBao Bridge with double thin wall high pier was taken as an example. Using FEA software MIDAS, bridge structural spatial finite element model was introduced. The software MIDAS was used to analyze the influence of radius of curvature on bridge's seismic behavior. The seismic response of the curve continuous rigid frame in the incentives under the different direction was investigated. Finally, some beneficial conclusions were obtained which can provide reference for the research and design of similar bridges.

INTRODUCTION

Western region of China is a mountainous zone, with high mountains and deep gorges, steep terrain. With the rapid development of highway projects, more and more long span curved continuous rigid frame bridges are necessary to build in order to satisfy the request of route direction and landform. Curved continuous rigid-frame bridge with high pier and long-span has been widely used in the construction of high-grade highways for the characteristics of great ability and good structural integrity, as well as the outstanding competitive advantages. Meanwhile, it has been used as one of the most popular types in high mountains and deep valley regions due to its excellent performance. However, domestic and foreign existing earthquake resistance design code only applies to ordinary bridges with relatively low piers and medium span. There is no code to guide the design of high pier and long-span bridges. In addition, few researchers have done the special theory analysis and research from the earthquake perspective. Therefore, it is necessary to research the dynamic characteristics seismic performance which has important theoretical and practical significance not only in accumulating data to instruct the engineering application but also in promoting the development of such bridges.

To analyze seismic response, a long span continuous rigid frame curved bridge--Wayabao Bridge with double thin wall high pier was taken as an example. Using FEA software MIDAS, bridge structural spatial finite element model was

introduced. The software MIDAS was used to analyze the influence of radius of curvature on bridge's seismic behavior. The seismic response of the curve continuous rigid frame in the incentives under the different direction was investigated.

ENGINEERING BACKGROUND

Wayaobao bridge located on Chongqing - Changsha section of the Bao mao Expressway, is a large-span curve prestressed Concrete continuous rigid frame bridge with high pier, design of vehicle load: Grade 1 Highway, speed: 80km/h, radius of horizontal curve is 2 500m. It' s spans consist of $71\text{m} + 3 \times 125\text{m} + 71\text{m}$, the root height of the main box girder is 7.5m, the middle height is 2.8m, roof width is 12.25m, backplane width is 6.5m, roof cantilever length is 2.875m. The end portion of the cantilever slab thickness is 15cm, thick root is 70cm. Root thick plate of the box girder is 100cm, across the floor thickness is 30cm, high beam and plate thickness from root to cross the change 2 times parabola, the web root thickness is 70cm, across the thickness is 40cm, using the linear change of three segmental box girder, box girder roof thickness is 28cm. Bored pile foundation is used in this bridge. High pier from left to right: No. 1:27m, No. 2:78m, No. 3:75m, No. 4:53m, asymmetric curved continuous rigid frame bridge. The right layout of the bridge is shown in Figure 1.

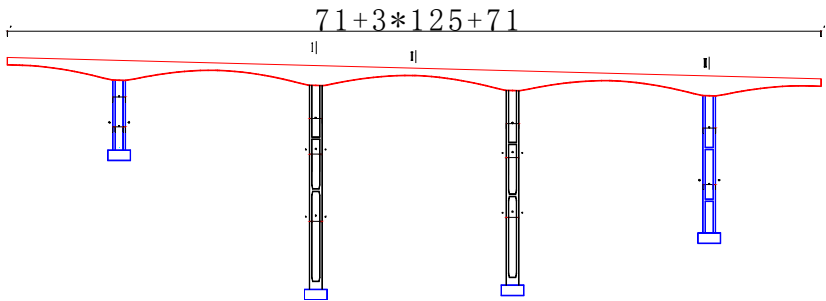
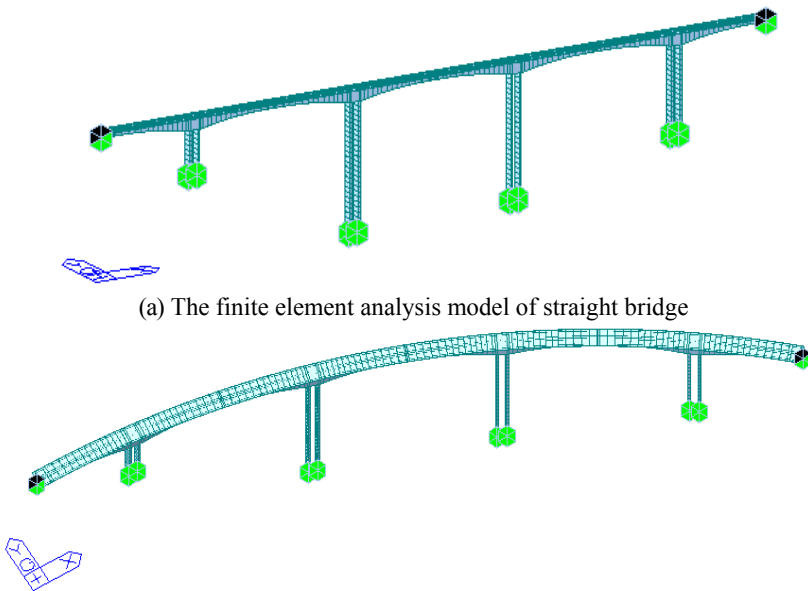


Figure 1. The Right Layout of the Wayaobao Bridge (Unit: cm)

FINITE ELEMENT MODEL

The large-scale general finite element program Midas/civil is used to build the bridge model, in order to compare and analyze the effects of straight continuous rigid frame bridge with curved continuous rigid frame bridge on seismic response and radius of curvature on the dynamic characteristics of bridge structure, established the calculation model of the straight bridge, the radius of curvature were calculated for model curve prestressed concrete 700m, 1000m, 1500m, 2000m, and 2500m continuous rigid frame bridge with Midas finite element software. In establishing the finite element computation model of main girder bridge, using variable section spatial beam element, the general beam element to simulate the main pier, pier and beam are connected by rigid elastic coupling in the implementation, not considering the interaction of pile and soil, the main bridge supports according to torsion bearing

simulation, the support of different simulation methods of bridge vibration the performance calculation results have great influence, therefore in the modeling will support the joint local coordinate vertical and horizontal shaft into the bridge curves in the tangential and normal , then node to the local coordinate system of horizontal, vertical constraint to the line displacement and torsion constraint. The whole bridge model is divided into 305 nodes, 294 units, as shown in Figure 2.



(a) The finite element analysis model of straight bridge

(b) The finite element analysis model of curved bridge

Figure 2. The Finite Element Analysis Model

SEISMIC ANALYSIS METHOD

Let $\ddot{y}_0(t)$, $\ddot{y}(t)$ denote the ground motion acceleration time histories, particle for movement relative to the ground acceleration time history, the vibration equation of single particle m under earthquake is:

$$m\ddot{y}''(t) + c\dot{y}(t) + ky(t) = -m\ddot{y}_0(t) \tag{1}$$

Among them, c, k respectively for the viscous damping coefficient and elastic recovery force coefficient. The differential equation theory, non-homogeneous equation (1) special solution available the Duhamel integral representation:

$$y(t) = -\frac{1}{\omega} \int_0^t \ddot{y}_0(\tau) e^{-\zeta\omega(t-\tau)} \sin \omega'(t-\tau) d\tau \tag{2}$$

Among them, ω 、 ω' were system without damping, a damping vibration frequency, $\omega' = \sqrt{1 - \zeta^2}$, ζ is damping ratio for the system; $\zeta = c/2 m\omega$. The damping ratio of bridge structure is very small, the difference between ω and ω' is very small, and the two can be approximately equal.

The inertial force acting on a particle is equal to the product of its quality times absolute acceleration:

$$P(t) = -m[\ddot{y}_0(t) + \dot{y}'(t)] \quad (3)$$

$$P(t) = ky(t) = -m\omega \int_0^t \ddot{y}_0(\tau) e^{-\xi\omega(t-\tau)} \sin \omega(t-\tau) d\tau \quad (4)$$

In the response spectrum analysis method, the seismic load on the structure of the seismic force $P(T)$ is the absolute value of the maximum. Let P denote the maximum absolute value of the horizontal seismic load:

$$P = m\omega \left| \int_0^t \ddot{y}_0(\tau) e^{-\xi\omega(t-\tau)} \sin \omega(t-\tau) d\tau \right|_{\max} \quad (5)$$

$$\begin{aligned} \text{令} : S_a &= \omega \left| \int_0^t \ddot{y}_0(\tau) e^{-\xi\omega(t-\tau)} \sin \omega(t-\tau) d\tau \right|_{\max} \\ &= \beta |\ddot{y}_0(t)|_{\max}, \end{aligned}$$

$$|\ddot{y}_0(t)|_{\max} = Kg。$$

$$P = m \cdot K \cdot \beta \cdot g = K \cdot \beta \cdot W \quad (6)$$

$$\beta = -S_a / |\ddot{y}_0(t)|_{\max} \quad (7)$$

$$\beta = \frac{2\pi}{T \cdot |\ddot{y}_0(t)|_{\max}} \left| \int_0^t \ddot{y}_0(\tau) e^{-\xi \frac{2\pi}{T}(t-\tau)} \sin \frac{2\pi}{T}(t-\tau) d\tau \right|_{\max} \quad (8)$$

$$[M]\{\ddot{y}(t)\} + [C]\{\dot{y}(t)\} + [K]\{y(t)\} = -[M]\{I_x\} \ddot{y}_0(t) \quad (9)$$

$$[C] = a_0[M] + a_1[K] \quad (10)$$

Among them, a_0, a_1 are constants.

SIMULATION RESULTS AND ANALYSIS

Under small earthquake, vibration spectrum method for seismic calculation of decomposition reaction is used, the following parameters input seismic load required: basic intensity: 7, site categories: II, importance correction factor: 1.3, the comprehensive influence coefficient: 0.33, the maximum period: 8 seconds, the above parameters can be input automatically" specifications for seismic design of Highway Engineering" (JTJ004-89) earthquake influence coefficient curve. Coordinate axis is defined as follows: X axis and the bridge at both ends of the connection in parallel, Y axis in the horizontal plane and perpendicular to the X axis, Z axis is a vertical axis. The X direction is called along the bridge, known as the Y direction transverse to the bridge, z directions is called the vertical direction of the bridge.

Calculation of curvature radius from 700 meters to infinity, which chooses the radius of curvature of 6 kinds of mode 700 m, 1000 m, 1500 m, 2000 m, 2500 m, and straight bridge were calculated. No. 1 pier is analyzed here.

Excitation along the bridge

(1) Analysis of displacement on the pier top

To drive along the bridge, along with the increase of the radius of curvature, the pier top tangential displacement value began to increase rapidly, the tangential displacement value decline as the curvature radius of 2500 m, the radius of curvature tends to ∞ , tangential displacement and upward trend; the radial displacement with the increase of the radius of curvature extra mum appears at 2500 meters the radius of curvature, tend to ∞ , displacement tends to 0. As shown in Figure 3.

(2) Analysis of shear of pier bottom

To drive along the bridge, along with the increase of the radius of curvature, cutting increases rapidly to the bottom of the pier shear, curvature radius of 700m, the tangential shear force value is 3827 kN, the radius of curvature increases to ∞ , tangential shear value is 5035kN,increased by 31.57%; the radial shear decreases, straight line bridge is 0. As shown in Figure 4.

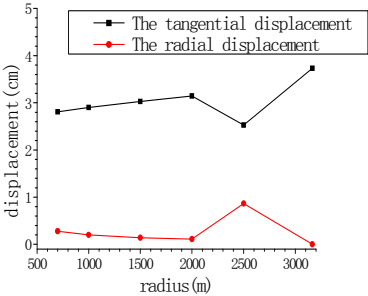


Figure 3. The Pier Displacement Curve Changes with the Radius of Curvature

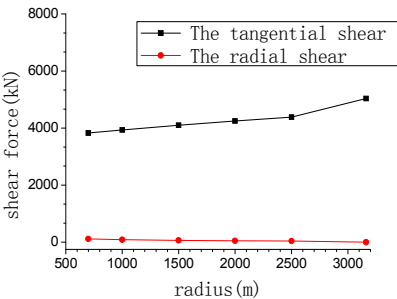


Figure 4. The Pier Shear Varies With the Radius of Curvature Curve

(3) Analysis of pier bottom moment To drive along the bridge, along with the increase of the radius of curvature, bending moments of pier bottom radial increases rapidly, the curvature radius is 700m, the bending moment value is 13814 kN*m, the radius of curvature increases to ∞ , bending moment value is 18282kN*m, increased by 33%; the bottom of the pier moment around the tangential gradual reduction. As shown in Figure 5.

(4) Analysis of girder root bending moment

To drive along the bridge, along with the increase of the radius of curvature, bending beam root radial increases, the radius of curvature of 700m, bending moment value is 13096kN *m, the radius of curvature increases to ∞ ,bending moment value is 17353kN *m, increased by 33%,bending moment of girder around the vertical root gradually reduced. As shown in Figure 6.

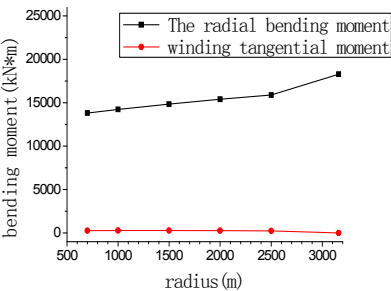


Figure 5. The Bottom of the Pier Moment Varies with the Radius of Curvature Curve

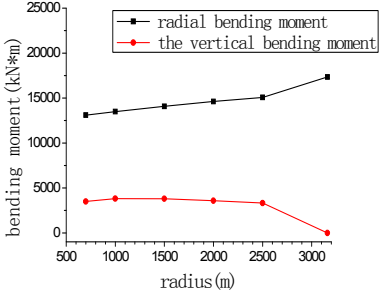


Figure 6. Curve Girder Root Bending Moment Varies With the Radius of Curvature

To sum up, when drive along the bridge, along with the increase of the radius of curvature, in addition to the pier top tangential displacement at 2500m decline, bending moment and the girder pier bottom cut roots to shear, pier bottom around the radial increase, therefore, under the same conditions, curved bridge earthquake resistance performance is better than the straight bridge.

Transverse Excitation

(1) Analysis of displacement on the pier top

Transverse excitation, with the increase of the radius of curvature, the pier top displacement first decreases, the sudden increase in 2500m, and then smaller. The pier top displacement decreases rapidly to cut. As shown in Figure 7.

(2) Analysis of shear of pier bottom

Transverse excitation, with the increase of the radius of curvature, the bottom of the pier radial shear small amplitude increases, the bottom of the pier shear decreases rapidly to the shear. As shown in Figure 8.

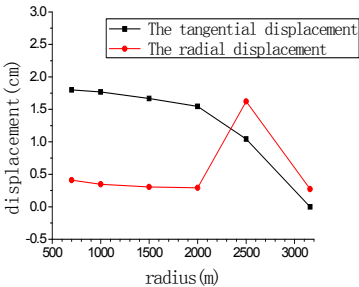


Figure 7. The Pier Displacement Curve Changes with the Radius of Curvature

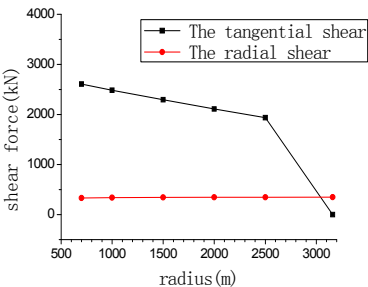


Figure 8. The Pier Shear Varies With the Radius of Curvature

(3) Analysis of pier bottom moment

Transverse excitation, with the increase of the radius of curvature, bending moment of pier bottom around the radial decreases rapidly; around the tangential bending moment decreases. As shown in Figure 9.

(4) Analysis of girder root bending moment

Transverse excitation, with the increase of the radius of curvature, bending beam root around the vertical first decreases and then small amplitude increases; radial bending moment decreases rapidly. As shown in Figure 10.

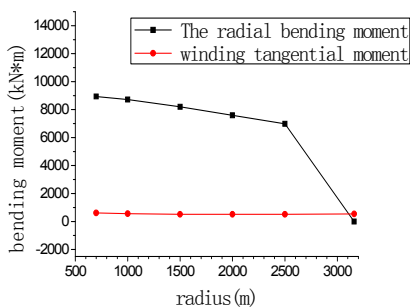


Figure 9. The Bottom of the Pier Moment Varies with the Radius of Curvature

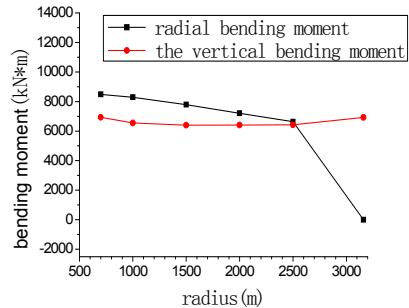


Figure 10. Curve Girder Root Bending Moment Varies with the Radius of Curvature

In summary, transverse excitation, with the increase of the radius of curvature, in addition to the root bending moment decreases, the rest of the main response values of the peak increase. Therefore, under the same conditions, the seismic performance is better than the curved bridge and straight bridge.

CONCLUSIONS

Response spectrum analysis method was used in this paper to study the dynamic response of high pier and long-span continuous rigid frame bridge with different curvature radius calculation, and the general law of peak response value with the changes of curvature radius. Conclusions are as follows:

- (1) To drive on the bridge, along with the increase of the radius of curvature, bending moment at the top of the tangential displacement, bending moment and the girder pier bottom cut roots to shear, pier bottom radial winding radial increased.
- (2) Transverse excitation, with the increase of the radius of curvature, in addition to the root bending moment decreases, the rest of the main response values of the peak increase.
- (3) For the displacement of pier top, curvature radius of 2500 meters is a turning point. Further study is needed.

Overall, under the same conditions, the seismic performance of the curved bridge is better than the straight bridge.

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Comparison Study on Seismic Performance of Different Structural Forms for Urban Expressway Viaducts

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ABSTRACT

Many structure forms can be chosen when designing an urban expressway viaduct. To get an optimal structure, a comparison study on the seismic performance of different structural forms for urban expressway viaducts should be done carefully. Taking multi-span simply-supported small box girders as a basic structure type, a double column pier and double pile cap structure (DCPDPC), a double column pier and single pile cap structure (DCPSPC), and a single column pier and single pile cap structure (SCPSPC) were chosen as substructures of the same bridge. Seismic performances, such as deformation capacity of bearings, bending capacity and shearing capacity of piers and piles in common earthquakes, and ductile capacity of piers under rare earthquakes of different structural forms were compared by the demand capacity ratio method. The optimal structures are discussed herein. It can be concluded that the structural form of DCPDPC and DCPSPC have better advantages than others, and the SCPSPC has poor seismic performance and should be carefully used when designing an urban expressway viaduct. This research provides good technical support for the conceptual design for urban expressway viaducts.

Key Words: Viaduct bridges; structure form; seismic performance; comparison study

INTRODUCTION

A viaduct is one of the most important forms of urban traffic congestion alternative structures in recent years. When designing an urban expressway viaduct, various structural forms can be selected (Chen 2013). Clearance under a bridge may be a key factor when choosing a bridge structure form. While affected by seismic geological conditions, seismic safety issues must be considered in the selection of structure forms. Since the 1980s, urban expressway viaducts have been damaged frequently in earthquakes. Cypress viaduct, which was built on U.S. 880 in 1957, collapsed when in

the 1989 Loma Prieta earthquake, even though the first phase of the reinforcement plan implemented by Caltrans. This included a vertical restraint device set between the longitudinal frames; however, due to a lack of steel bars in the beam-column node and transverse stirrups, the anchorage length was generally not enough, so a section of the upper deck in the 800m-long viaduct slumped in the lower deck due to pier fracture, as shown in Figure 1. In 1995, the Kobe earthquake occurred in Japan. Eighteen single piers on the Hanshin viaduct experienced shearing failure, and the 635m-long superstructure overturned (Figure 2). This bridge is a cantilevered T structure with hanger; the pier is a 3.1m wide, 15m high reinforced concrete cylinder pier. From the pier bottom, 30 percent longitudinal reinforcement (60D35, SD345 reinforcement) was been shearing breaks. Under the top of the pier, stirrups (D16, SD345 reinforcement) had 200mm spacing; in the middle section (4.2m) is 300mm spacing. The main reason was that longitudinal reinforcement cut off prematurely and the stirrups were inadequate.



Figure 1. Cypress viaduct, collapsed in Loma Prieta earthquake.



Figure 2. Hanshin viaduct, overturned in Kobe earthquake.

Earthquake damage to numerous structures showed that improper reinforcement was the main factor that caused urban viaduct collapse. However, it cannot be ignored that structural stiffness and periodic changes led by superstructure and substructural changes caused changes in the seismic response of the structure. A reasonable urban viaduct structure that considers seismic conditions is worthy of further exploration and research.

Guan (2011) conducted research on urban viaduct ductility seismic systems and seismic isolation systems. His research shows that the seismic responses of substructure can be easily reduced by a seismic isolation system. In addition, the overall project cost was reduced significantly, so investment costs can be saved. The isolation system is a good selection and should be popularized. Although the concept has caught attention gradually, seismic isolation design is still difficult to master by the general engineer, and in recent days the quality of a seismic isolation device is also difficult to guarantee. Comparison of seismic performance of common structure should be done immediately to obtain a deep understanding of the systems, and then better decisions can be made when designing a viaduct.

The authors investigated city viaduct superstructure and substructure selection to determine the relative size of seismic response in a city viaduct under common and rare earthquake excitation. By good understanding the seismic performance of various structures, a reasonable seismic system can be selected.

GENERAL SITUATION OF SAMPLE BRIDGE

A 4×30m simply-supported and continuous-deck bridge, generally used as a part of a viaduct bridge, was used as a sample bridge, and its substructure was changed to different structural forms for comparison. The width of the bridge was 24.8m and consisted of seven small box girders. According to the *General Code for Design of Highway Bridges and Culverts in China*, the design vehicle load is Highway-I type. A total of 14 rubber bearings (GYZ300×77mm) on top of every middle pier and 14 rubber bearings (GYZF₄300×76 mm) on the top of every transition pier were chosen to support the bridge. The substructure was composed of double column pier and single pile cap structure (DCPSPC). The average height of the columns was eight meters. The cross-section dimensions of the rectangular column were 1.8m×1.5m. The piers were supported on Group Piles foundations, of which the rectangular-shaped pile cap was 7.64m long, 8m wide and 2.5m high. Below are 4 piles, with a uniform diameter of 1.8m and uniform lengths exceeding 11m. The upper area of the seismic block stops was 85cm×50cm, the bottom area was 85cm×65cm, and the height of the block was 60cm. According to *Specifications of Earthquake Resistant Design for Highway Engineering in China*, the design seismic intensity of this bridge is level VIII, and the soil type at the natural scouring depth is Type II. The characteristic site period is 0.4s. The typical section can be seen in Figure 3.

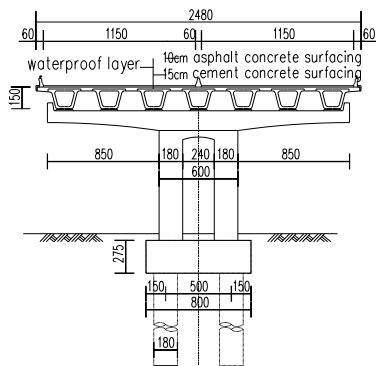


Figure 3. Typical section of the sample bridge.

A literature review shows that three structure forms, such as the double column pier and double pile cap structure (DCPDPC), double column pier and single pile cap structure (DCPSPC), and single column pier and single pile cap structure (SCPSPC) usually can be chosen when designing an urban viaduct bridge. The mechanical behavior and advantages and disadvantages are described below:

- (1) SCPSPC – this structure form can provide the maximum traffic capacity and make full use of the land because the transverse dimension of substructure is limited. The cantiliver part of the cap beam is very long, so the unfavorable force will show at the cantilever root. Because there is less redundancy, the damage will concentrate at the bottom of the pier when an earthquake occurs.
- (2) DCPSPC – to improve mechanical behavior, double columns are used in

this type of bridge. For providing maximum traffic capacity and to make full use of the land, the transverse distance of the column should not exceed 8m. The unfavorable force will also show at the cantilever root. However, because of its redundancy, the damage will concentrate at the top and bottom of the piers when an earthquake occurs.

(3) DCPDPC – this kind of bridge looks like a portal pier or frame pier. Because the transverse distance is increased, the mechanical behavior can be improved. The damage level will be decreased more and can be easily repaired when an earthquake occurs. The disadvantage of this structure is that two separate group pile foundations will be used, and project quantity of the basis will be increased.

MODEL ANALYSIS

To consider the effects of adjacent girders on the transition pier, the adjacent girders were modeled by single beam elements. A grillage method model was used to model the concerning girders. For every part, the cap beam, piers, and piles were modeled with elastic beam elements. The pile cap was modeled by the thickness plate element. The elastic springs were used to model the soil- structure interaction effects. Three models using different substructure were built in this research, as shown in Figure 4.

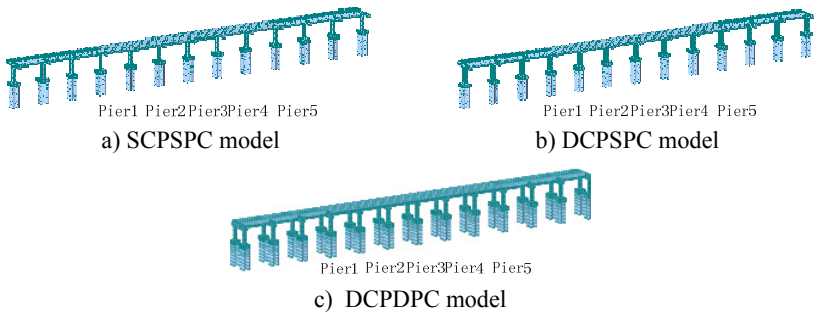


Figure 4. FEA model of typical viaduct bridges.

Much earthquake damage shows that the collision between the two separation parts cannot be neglected. The longitudinal collisions between the deck and the cap beam and transverse collisions between the restraining block and main girder were considered in this research. Jankowski (2005) shows that if choosing reasonable parameters, the Kevin model can give results similar to experiments. In this research, the Kevin model was used, which is composed of a dashpot element and a gap element, as shown in Figure 2. The collision force can be transferred by the spring element. The damper in the Kevin model can model the energy loss, and the gap can be used to model the initial clearance between the deck and the cap beam or between the restraining block and the deck. The collision force can be calculated as

$$F = \begin{cases} k_k(d + d_o) + c_k v & d + d_o < 0 \\ 0 & d + d_o > 0 \end{cases} \quad (1)$$

Where k_k is the stiffness of the spring; c_k is the damping constant; v is velocity; d is the deformation of the spring; and d_o is initial clearance. Based on the the law of

conservation of energy, the damping constant c_k can be caculated by restitution coefficient e :

$$c_k = 2\zeta \sqrt{k_k \frac{m_1 m_2}{m_1 + m_2}} \tag{2}$$

$$\zeta = - \frac{\ln e}{\sqrt{\pi^2 + (\ln e)^2}} \tag{3}$$

On the other hand, the ductility earthquake-resistance system is generally used in common bridges. When an earthquake occurs, the bottom part or top part of the RC columns begins in a plastic state; the stiffness can be decreased, and the structure period can be prolonged to be far from the earthquake excellence energy frequency. The ductility earthquake-resistance system can be modeled as one or several plastic hinges are formed at some potential plastic hinge regions in some elements. To capture the plastic state, the fiber elements (Taucer 2005) were used in the bottom part or top part of the pier, as shown in much research. The collision and the ductility were modeled in this paper, as can be seen in Figure 5.

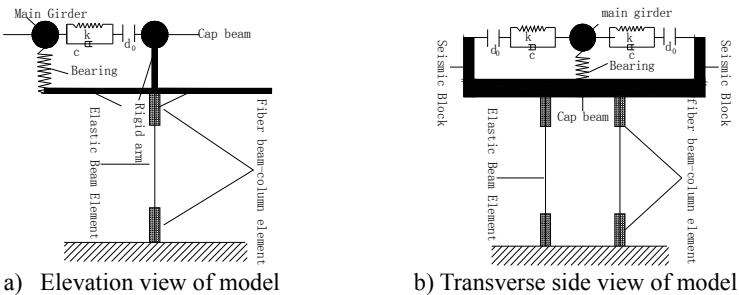


Figure 5. Model of collision and ductility of bridge.

GROUND MOTION

According to *Guidelines for Seismic Design of Highway Bridges*, the E1 acceleration response spectrum at the site with an exceedance probability of 63 percent in 100 years and the E2 acceleration response spectrum with an exceedance probability of 2 percent in 100 years were calculated, as shown in Figure 6.

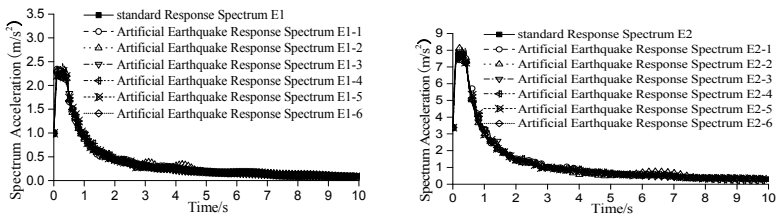


Figure 6. Acceleration response spectrum of (a) E1-level and (b) E2-level

Taking the above acceleration response spectrum as the target spectrum, 12 artificial accelerograms were synthesized, as shown in Figure 7. The response spectrum of these artificial waves can be seen Figure 6. These accelerograms can be applied to do time history analysis for the FEA model.

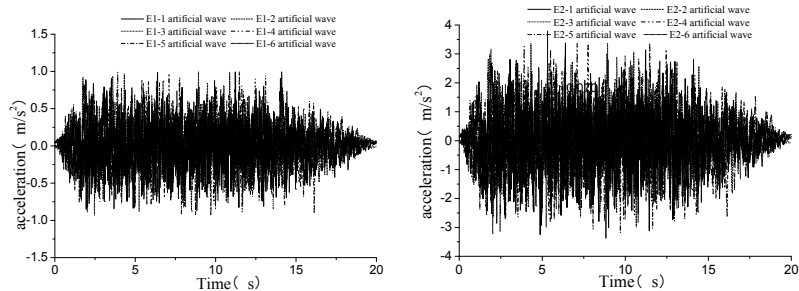


Figure 7. Artificial accelerograms of (a) E1-level and (b) E2-level

COMPARISON STUDY OF DIFFERENT STRUCTURAL FORMS

Seismic performance of DCPSPC structure under E1-level earthquake. The seismic performance of different structures such as the DCPDPC, DCPSPC, and SCPSPC were discussed in this research. Table 1 and Table 2 show the seismic demand and capacity at the bottom of the pier excited by an E1 and an E2 earthquake. The result shows that the FEA model is fine, by which reasonable and reliable seismic performance can be obtained.

Table 1. Capacity/demand ratio of key sections under E1-level earthquake.
Longitudinal Direction Capacity/Demand Ratio Analysis

Key section	Shear force demand (kN)	Shear force capacity (kN)	Capacity/demand ratio	Bending moment demand (kNm)	Bending moment capacity (kNm)	Capacity/demand ratio
Pier1 bottom	275	5679	20.58	3779	13485	3.6
Pier2 bottom	488	5679	11.64	3984	13485	3.4
Pier3 bottom	485	5679	11.72	4083	13485	3.3
Pier4 bottom	523	5679	10.85	4260	13485	3.2
Pier5 bottom	228	5679	24.88	3447	13485	3.9

Table 2. Capacity demand ratio of key sections under E2-level earthquake.
Longitudinal Direction Capacity/Demand Ratio Analysis

key section	Shear force demand (kN)	Shear force capacity (kN)	Capacity/demand ratio	Maximum section curvature demand (1/m)	Maximum section curvature capacity (1/m)	Capacity/demand ratio
Pier1 bottom	343	2820	8.22	0.00128	0.00542	4.2
Pier2 bottom	439	2820	6.41	0.00191	0.00542	2.8

Pier3 bottom	433	2820	6.50	0.00200	0.00542	2.7
Pier4 bottom	424	2820	6.64	0.00198	0.00542	2.7
Pier5 bottom	339	2820	8.31	0.00124	0.00542	4.4

Comparison study under E1-level earthquake. Figure 8 shows capacity/demand ratio (C/D Ratio) at the pier bottom of three different models (E1-X is the result of longitudinal direction under an E1-level earthquake, E1-Y is the result of transverse direction under an E1-level earthquake). Figure 9 shows the bending moment and shear force capacity/demand ratio at the top of the most unfavorable single pile. Figure 10 shows the capacity/demand ratio of the maximum bearing deformation and the relative displacement between the deck and piers.

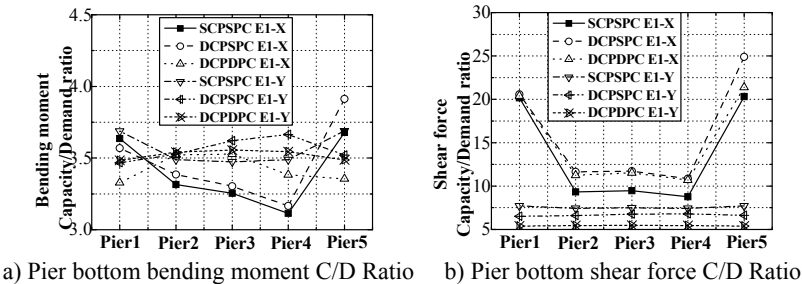


Fig. 8 C/D Ratio of pier under E1-level earthquake.

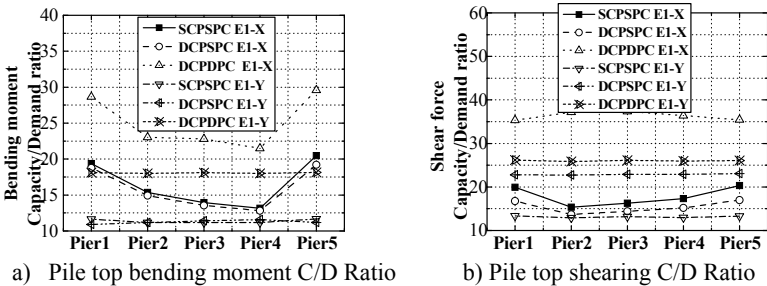


Figure 9. C/D Ratio of pile under E1-level earthquake.

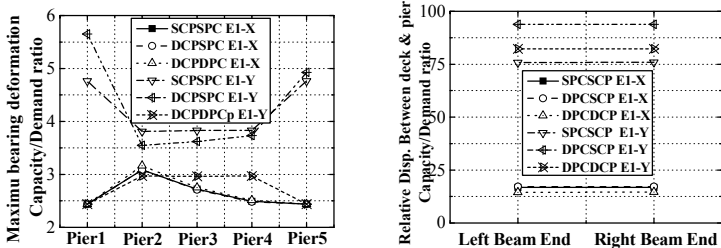


Figure 10. C/D Ratio of maximum bearing deformation (a) and relative displacement (b) between deck and piers under E1-level earthquake.

From above Figures 8 to 10, it can be concluded that:

- (1) As shown in Figure 8, DCPDPC and DCPSPC structures have a higher bending moment C/D ratio than SCPSPC structures in the longitudinal or transverse directions. By using the sliding bearing in the longitudinal direction on top of the transition pier, transition piers have a higher shear force C/D ratio than other piers. Because all bearing are fixed in the transverse direction, the shear force C/D ratio of all three structures are similar in the transverse direction, and the DCPDPC and DCPSPC structures have a little higher shear force C/D ratio than the SCPSPC structure.
- (2) As shown in Figure 9, the piles of the DCPDPC structure have a higher bending moment and shear force C/D ratio than those in the DCPSPC structure and the SCPSPC structure in the longitudinal or transverse directions. The reason is that the DCPDPC structure has more piles than the other two structures.
- (3) As shown in Figure 10(a), because in the longitudinal direction, three structures have similar stiffness, the maximum bearing deformation C/D ratios are very close. In the transverse direction, the ratios of SCPSPC and DCPSPC are higher than that of DCPDPC.
- (4) As shown in Figure 10(b), the relative displacement between the deck and piers C/D ratios are very close in the longitudinal direction. In the transverse direction, the relative displacement C/D ratios of SCPSPC and DCPSPC are higher than those of DCPDPC.

Comparison study under E2-level earthquake. To investigate the seismic performance of the whole structure under an E2-level earthquake, the C/D ratio of the maximum section curvature at the bottom or top of the piers were calculated under the most unfavorable axis forces, as shown in Figure 11. (E2-X is the result of the longitudinal direction under an E2-level earthquake, and E2-Y is the result of the transverse direction under an E2-level earthquake). Figure 12 shows the shear force and bending moment C/D ratio at the top of the most unfavorable single pile. Figure 13 shows C/D ratios of maximum bearing deformation and the relative displacement between deck and piers.

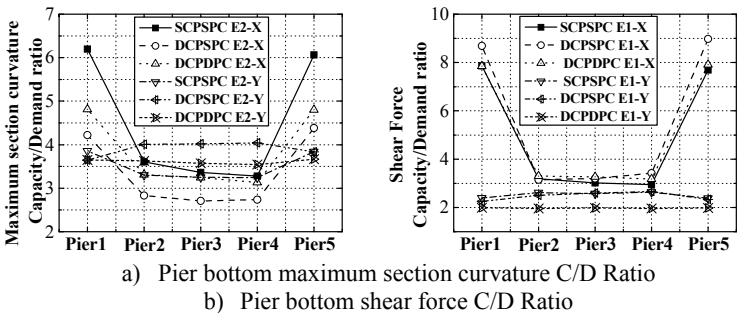


Figure 11 C/D Ratio of pier under E2-level earthquake.

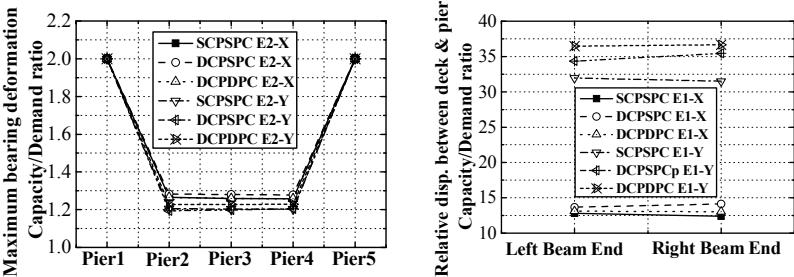
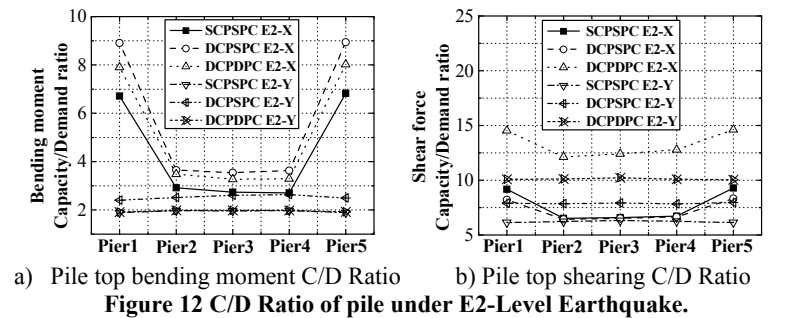


Figure 13 C/D Ratio of maximum bearing deformation (a) and relative displacement (b) between deck and piers under E2-level earthquake.

From above Figures 11 to 13, it can be concluded that:

- (1) As shown in Figure 11, SCPDPC structures have a higher maximum section curvature C/D ratio than DCPSPC and DCPDPC structures in the longitudinal direction. In the transverse direction, DCPSPC structures have a higher maximum section curvature C/D ratio than the other two structures. DCPDPC and DCPSPC structures have a little higher shear force C/D ratio than SCPSPC structures.
- (2) Under an E2-level earthquake, as shown in Figure 12, the piles of the DCPDPC structure have a higher bend moment and shear force C/D ratio than those in the DCPSPC and SCPSPC structures in the longitudinal or transverse directions. The reason is that DCPDPC structures will have more piles than the other two structures.
- (3) As shown in Figure 13(a), considering the collision between the girder and the cap beam in the longitudinal and transverse directions, the maximum bearing deformation C/D ratios of the three structures are very close.
- (4) As shown in Figure 13(b), the relative displacement between the deck and piers C/D ratios are very close in the longitudinal direction. In the transverse direction, the relative displacement C/D ratios of DCPSPC and DCPDPC are higher than those of SCPSPC.

CONCLUSIONS

From this study, it can be concluded that:

(1) When designing an urban viaduct bridge, DCPSPC is an alternative proposal. It can provide the maximum traffic capacity and make full use of the land; at the same time, damage will concentrate at the top or bottom of the piers when an earthquake occurs. In general, DCPSPC is a reasonable choice.

(2) From the seismic demand and capacity ratios of the bottom of the pier in the DCPSPC structure excited by E1 and E2 earthquakes, the model used in this paper looks reasonable and accurate and can be used to do a comparison study.

(3) Under an E1 earthquake, although it has a higher stiffness, the seismic demands were not increased sharply compared with other structures. DCPDPC and DCPSPC structures have higher flexural performance in the pier and piles than SCPSPC structures. The DCPDPC structure has more superiority than others in bearing deformation and the relative displacements between deck and piers.

(4) Under an E2 earthquake, when considering the collision between the girder and the cap beam in the longitudinal and transverse directions, DCPDPC and DCPSPC structures have no clear superiority in ductility capacity than the other two structures. However, for relative displacements, DCPDPC and DCPSPC structures have some superiority over others.

ACKNOWLEDGMENTS

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Health Monitoring Strategy for Bridge Considering Vehicle-bridge Interaction

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ABSTRACT

The monitoring strategy for bridges considering signal-to-noise ratio and vehicle-bridge interaction is proposed in order to discard a large amount of low signal-to-noise ratio data or normal data. The composition of the vibration signal is discussed, and the reasonable and accurate monitoring threshold should be determined according to vehicles-bridge interaction dynamics. A time-varying dynamic vehicle group model, which includes main uniform parameters and coupling mass, is established. The dynamic coupling equations for bridge and car groups are established, and the analytical expression is given. The health monitoring for a simply-supported beam is discussed. The dynamic response under different uniform mass, speed, and vibration frequency of the car group is analyzed based on the vehicle-bridge interaction theory. The monitoring threshold is determined according to simulated results and measured signals.

INTRODUCTION

In the long service life and continuous operation process, bridges suffer resistance attenuation and damage accumulation under the comprehensive effect of vehicle load and environment effects such as wind load and rain load. Moreover, serious abrupt destruction may occur under the dynamic action of earthquakes and hurricanes, affecting the reliability of bridges. By placing many different types of sensors on important bridges, the health status of the bridges can be monitored on line through a health monitoring system in order to determine the degree of damage, which not only can ensure the operational safety of the bridge and raise the level of management but also determine the deterioration of the bridge. Hence, bridge health monitoring became an important research field in bridge engineering, and health monitoring systems have been applied on many important bridges worldwide. However, bridge health monitoring is a comprehensive and imperfect technology, with disadvantages such as sensor performance and signal transmission, data mining and processing, safety assessment theory, software development, and not fully considering the environmental changes.

Due to complex operation environments and many key parts on bridges, many sensors need to be arranged for bridge monitoring. In the progress of monitoring, mass data are collected; however, data acquisition, transmission, storage, and analysis are time-consuming. The effectiveness of monitoring can be reduced if the monitoring data

are not dealt with properly, which is one of the bottlenecks restricting the development of bridge health monitoring. To overcome these problems, the data compression, data storage, and monitoring strategy can be optimized. Zhang (2006) proposed a data compression algorithm based on lifting wavelet transform, which can realize high compression efficiency, but it is a loss compression algorithm. Bao (2009) proposed a Bayesian compressive sampling method which has good accuracy and denoising ability, but it is more suitable for narrow band process. Lin (2010) surmised that the integrity, calling efficiency, and automation management of the data in the process of structural health monitoring should be considered. The problem of data processing and efficiency are solved through a data organization strategy in order to fully optimize the data management system.

The current research on a health monitoring strategy is not enough. Frangopol (2011) built a health monitoring framework and computing platform that considers the whole life cycle of the bridge in order to ensure the safety and adjust the maintenance scheme under different technical conditions. Kripakaran (2009) put forward that different monitoring strategies can be used for different goals in bridge health monitoring to ensure normal operation and monitoring performance in case of abnormal data. The performance, size, and cost of a bridge should be determined by the scale of the bridge, the monitoring importance, and reasonable project cost. Combining health monitoring and routine detection, a real-time and timely monitoring model is the basic principle for improving the efficiency of a monitoring system. In addition, the specific form of a bridge, the static and dynamic characteristics, and the accuracy and stability of the equipment should be also considered. Therefore, bridge monitoring strategies considering the signal-to-noise ratio and the vehicle-bridge interaction are proposed in this paper.

MONITORING STRATEGY CONSIDERING NOISE RATIO

The precision of a monitoring signal is one of the key factors determining the monitoring level. The measured signal received through the sensor and data acquisition instrument includes three types: (1) structural vibration signal produced by the traffic load; it has large amplitude, which can fully reflect the dynamic characteristics of structure; (2) environmental vibration signal caused by the natural environment; it is produced by a tiny vibration of the site, wind, earthquakes, and other natural roles; such a signal has a small amplitude and the randomness, structure dynamic characteristics, and frequency band are usually wide; (3) electromagnetic interference signals or noise signals produced by electromagnetic interference of natural and electrical and electronics equipment; such a signal seriously reduces the authenticity and validity of data and should be eliminated.

The important part in monitoring is removing electromagnetic interference signals or noise signals to highlight the vibration signal. Typically, various filtering techniques can partially attenuate electromagnetic interference signals, but it cannot completely remove noise. In addition, a general filtering technique cannot realize filters in real time, and the calculation amount is large. Hence, mass data processing and optimization of the monitoring strategy must be adopted in actual data monitoring and analysis in order to realize the most effective monitoring.

In this paper, a health monitoring strategy for bridges considering a signal-to-noise ratio and vehicle-bridge interaction is proposed as follows. The health monitoring demand is presented according to the structure state, form, and economic condition, and the goal is to assure the structural safety of the entire life cycle. Appropriate monitoring instruments and software system were chosen to achieve a combination of healthy monitoring and conventional detection and to achieve the combination of real-time monitoring and timely monitoring. The dynamic characteristics of the bridge were estimated in the ambient vibration and traffic loads according to traffic data such as vehicle number, weight, and speed from the monitoring system, when the performance parameters in the monitoring instrument, especially the signal-to-noise ratio, meet the requirement. Multilevel thresholds were defined for different traffic cases and safety states for evidence to determine whether to start the signal filter and damage detection. After damage detection was executed, the analysis model and monitoring strategy were regulated, and the maintenance scheme was presented. This strategy has advantages such as abandoning large low signal-to-noise ratios or no abnormal data and saving the computing resources, thus achieving the targeted monitoring based on theory and analysis.

EQUATIONS OF VEHICLE GROUP-BRIDGE COUPLED MOTION

Before the health monitoring of bridges, dynamic characteristics and traffic loads of the bridge should be investigated and analyzed, the characteristics of vehicle-bridge coupling need be considered, and then the damage detection can be carried out. In this paper, vehicle group is equivalent to the time-varying dynamic model that has the body portion and the coupling distributed mass; the dynamic equilibrium equation and analytical expression regarding the interaction of the bridge structure and the moving vehicle group are established.

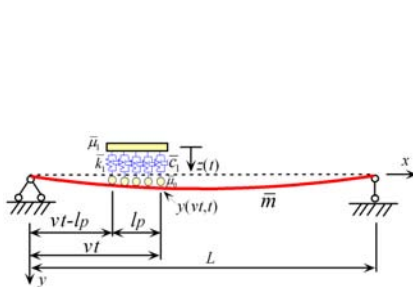


Figure 1. Interaction scheme.

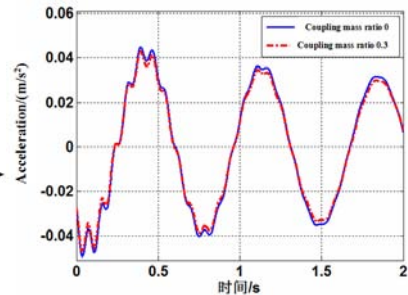


Figure 2. Interaction of vehicle–structure.

The main deformation of general bridge structure under the action of vertical vibration is bending deformation, and the shear deformation and rotary inertia cross-section around the neutral axis can be ignored. As shown in Figures 1 and 2, the bridge is assumed to be an equal section, so the flexural rigidity D is a constant. Vehicle group and structure coupling and dynamic response can be obtained by direct methods for solving differential equations under the premise of putting the vehicle group as a

uniformly moving mass load. The extrinsic motivation on the beam is caused by vehicle group, assuming the group consists of the main body uniform mass density $\bar{\mu}_1$, the main body stiffness $\bar{k}_1$, the main body damping $\bar{c}_1$, and coupling uniform mass density $\bar{\mu}_0$. Assuming the deflection of the beam is $y(x, t)$, the displacement of vehicle group of main body is $z(t)$, the distribution length is l_p , and the vehicle group move at a constant speed of v . The dynamic equation for vehicle group body is

$$m_1 \ddot{z}(t) + \bar{k}_1 (z(t) - y(x, t)) + \bar{c}_1 (\dot{z}(t) - \partial y(x, t) / \partial t) = 0 \quad (1)$$

The dynamic equation of the bridge on the vehicle group pedestrian excitation is

$$D \partial^4 y(x, t) / \partial x^4 + \bar{m} \partial^2 y(x, t) / \partial t^2 + c \partial y(x, t) / \partial t = p(x, t) \quad (2)$$

The dynamic loads $p(x, t)$ on the bridge caused by vehicle group quality involves the gravity load of the vehicle group mass on the bridge $p_G(x, t) = (\bar{\mu}_0 + \bar{\mu}_1)g$. The inertial force are generated by vehicle group $p_I(x, t) = -\bar{\mu}_0 \partial^2 y(x, t) / \partial t^2$, the side effects are generated by vehicle group stiffness $p_S(x, t) = \bar{k}_1 [z(t) - y(x, t)]$, and the side effects are generated by vehicle group damping $p_D(x, t) = \bar{c}_1 [\dot{z}(t) - \partial y(x, t) / \partial t]$. $p(x, t) = [p_G(x, t) + p_I(x, t) + p_S(x, t) + p_D(x, t)] \{H[x - (Vt - l_p)] - H(x - Vt)\}$. $H(t)$ is a unit step function. The above equation can be solved by the mode-superposition method. The transformation expression is shown as

$$y(x, t) = \sum_{i=1}^{\infty} q_i(t) \cdot \phi_i(x) \quad (3)$$

where $q_i(t)$ is the generalized modal coordinates; $\phi_i(x)$ is the vibration mode function.

Substituting Eq. (3) into Eq. (2), each item is multiplied the n -order vibration mode function. Then, the integral along the bridge considering the orthogonality of the vibration mode, the dynamic equations of bridge and vehicle group coupling systems can be obtained with the vibration mode decomposition method and jointing Eq. (1). The n -order modal response can be obtained by the stepwise integration method under the influence of the vehicle group. For multi-span continuous bridges or other types of suspension and cable-stayed bridges, since the vibration mode is complex, a related vibration mode needs to be obtained through modal analysis by applying the finite element method. For a simply supported beam, the following analytical method can be taken. The n -order vibration of simply supported beam is $\phi_n = \sin \frac{n\pi v}{L} (t - \frac{l_p}{2v})$,

integrating along the bridge, the result is

$$p_G(x, t) = \int_{vt-l_p}^{vt} (\bar{\mu}_1 + \bar{\mu}_2)g \sin \frac{n\pi x}{L} dx = \frac{2(\bar{\mu}_0 + \bar{\mu}_1)gL}{n\pi} \sin \frac{n\pi l_p}{2L} \sin \frac{n\pi v}{L} (t - \frac{l_p}{2v}) \quad (4)$$

$$p_I(x, t) = -\sum_{i=1}^{\infty} \bar{\mu}_0 \ddot{q}_i(t) \int_{vt-l_p}^{vt} \sin \frac{i\pi x}{L} \sin \frac{n\pi x}{L} dx = -\frac{\bar{\mu}_0 L}{\pi} \sum_{i=1}^{\infty} \ddot{q}_i(t) \Phi_{in} \quad (5)$$

$$\Phi_{in} = \frac{1}{i-n} \left[\cos \frac{(i-n)\pi v}{L} (t - \frac{l_p}{2v}) \sin \frac{(i-n)\pi l_p}{2L} \right] - \frac{1}{i+n} \left[\cos \frac{(i+n)\pi v}{L} (t - \frac{l_p}{2v}) \sin \frac{(i+n)\pi l_p}{2L} \right]$$

$$p_s(x, t) = \frac{2\bar{k}_1 L z(t)}{n\pi} \sin \frac{n\pi l_p}{2L} \sin \frac{n\pi v}{L} \left(t - \frac{l_p}{2v}\right) - \frac{\bar{k}_1 L}{\pi} \sum_{i=1}^{\infty} q_i(t) \Phi_{in} \quad (6)$$

$$p_D(x, t) = \frac{2\bar{c}_1 L \dot{z}(t)}{n\pi} \sin \frac{n\pi l_p}{2L} \sin \frac{n\pi v}{L} \left(t - \frac{l_p}{2v}\right) - \frac{\bar{c}_1 L}{\pi} \sum_{i=1}^{\infty} \dot{q}_i(t) \Phi_{in} \quad (7)$$

Order $\omega_n = n^2 \pi^2 \sqrt{EI/\bar{m}}/L^2$ to n -order circular frequencies of the structure, $c_n = 2\bar{m}\xi_n \omega_n$ is n -order vibration damping. Dividing both sides of Eq. (2) by $\bar{m}L/2$ and organizing the equation, then the standard form of beam-slab structure dynamic equilibrium equation for the n -order vibration mode under vehicle group excitation is obtained.

$$\begin{aligned} [\ddot{q}_n(t) + \frac{2\bar{\mu}_0}{\bar{m}\pi} \sum_{i=1}^{\infty} \ddot{q}_i(t) \Phi_{in}] + \left[2\xi_n \omega_n \dot{q}_n(t) + \frac{2\bar{c}_1}{\bar{m}\pi} \sum_{i=1}^{\infty} \dot{q}_i(t) \Phi_{in} \right] + \left[\omega_n^2 q_n(t) + \frac{2\bar{k}_1}{\bar{m}\pi} \sum_{i=1}^{\infty} q_i(t) \Phi_{in} \right] \\ - \frac{4}{n\bar{m}\pi} \sin \frac{n\pi l_p}{2L} \phi_n \left[\bar{k}_1 z(t) + \bar{c}_1 \dot{z}(t) \right] = \frac{4(\bar{\mu}_0 + \bar{\mu}_1)g}{n\bar{m}\pi} \sin \frac{n\pi l_p}{2L} \phi_n \end{aligned} \quad (8)$$

For a simply supported girder bridge, if the generalized coordinates are taken n items, overall freedom will be simplified to n from an infinite number, system motion equations of order n is converted from Eq. (1) into

$$\bar{\mu}_1 \ddot{z}(t) + \bar{c}_1 \dot{z}(t) + \bar{k}_1 z(t) - \bar{c}_1 \sum_{i=1}^n \dot{q}_i(t) \cdot \phi_i(x) - \bar{k}_1 \sum_{i=1}^n q_i(t) \cdot \phi_i(x) = 0 \quad (9)$$

Uniting Eq. (8) with Eq. (9), the dynamic equations of the bridge vehicle group coupling system can be obtained. For a simply supported girder bridge, if the displacement series is taken as n items, then the generalized degrees of freedom of the bridge are n , including the freedom of the vehicle group subject, so the $n+1$ order matrix expression of System equation of motion is as follows:

$$\mathbf{M}\ddot{\mathbf{U}} + \mathbf{C}\dot{\mathbf{U}} + \mathbf{K}\mathbf{U} = \mathbf{F}(t) \quad (10)$$

The generalized displacement vector is $\mathbf{U} = [q_1, q_2, \dots, q_n, z]^T$. The generalized mass matrix $\mathbf{M}$, damping matrix $\mathbf{C}$, stiffness matrix $\mathbf{K}$ and force vector $\mathbf{F}(\mathbf{t})$ are

$$\mathbf{M} = \begin{bmatrix} 1 + \rho_M \Phi_{11} & \rho_M \Phi_{12} & \cdots & \rho_M \Phi_{1n} & 0 \\ \rho_M \Phi_{21} & 1 + \rho_M \Phi_{22} & \cdots & \rho_M \Phi_{2n} & 0 \\ \vdots & \vdots & \ddots & \vdots & \vdots \\ \rho_M \Phi_{n1} & \rho_M \Phi_{n2} & \cdots & 1 + \rho_M \Phi_{nn} & 0 \\ 0 & 0 & \cdots & 0 & \bar{\mu}_1 \end{bmatrix} \quad (11)$$

$$\mathbf{C} = \begin{bmatrix} 2\xi_1 \omega_1 + \rho_C \Phi_{11} & \rho_C \Phi_{12} & \cdots & \rho_C \Phi_{1n} & -\rho_C \phi_1 \\ \rho_C \Phi_{21} & 2\xi_2 \omega_2 + \rho_C \Phi_{22} & \cdots & \rho_C \Phi_{2n} & -\rho_C \phi_2 \\ \vdots & \vdots & \ddots & \vdots & \vdots \\ \rho_C \Phi_{n1} & \rho_C \Phi_{n2} & \cdots & 2\xi_n \omega_n + \rho_C \Phi_{nn} & -\rho_C \phi_n \\ -\bar{c}_1 \phi_1 & -\bar{c}_1 \phi_2 & \cdots & -\bar{c}_1 \phi_n & \bar{c}_1 \end{bmatrix} \quad (12)$$

$$K = \begin{bmatrix} \omega_1^2 + \rho_K \Phi_{11} & \rho_K \Phi_{12} & \cdots & \rho_K \Phi_{1n} & -\rho_K \phi_1 \\ \rho_K \Phi_{21} & \omega_2^2 + \rho_K \Phi_{22} & \cdots & \rho_K \Phi_{2n} & -\rho_K \phi_2 \\ \vdots & \vdots & \ddots & \vdots & \vdots \\ \rho_K \Phi_{n1} & \rho_K \Phi_{n2} & \cdots & \omega_n^2 + \rho_K \Phi_{nn} & -\rho_K \phi_n \\ -\bar{k}_1 \phi_1 & -\bar{k}_1 \phi_2 & \cdots & -\bar{k}_1 \phi_n & \bar{k}_1 \end{bmatrix} \quad (13)$$

$$F(t) = [\rho_F \phi_1, \rho_F \phi_2, \dots, \rho_F \phi_n, 0]^T \quad (14)$$

where $\rho_M = \frac{2\bar{\mu}_0}{\bar{m}\pi}$, $\rho_C = \frac{2\bar{c}_1}{\bar{m}\pi}$, $\rho_K = \frac{2\bar{k}_1}{\bar{m}\pi}$, $\rho_F = \frac{4(\bar{\mu}_0 + \bar{\mu}_1)g}{n\bar{m}\pi} \sin \frac{n\pi l_p}{2L}$, $\Phi_{nn} = \phi_n \phi_n$, and $\phi_n = \sin \frac{n\pi v}{L} (t - \frac{l_p}{2v})$

The coefficient Φ_{nn} in coupling mass and the system dynamic equations varies with the vehicle group mobile on the bridge, which makes the system equations become order differential equations of the time-varying system in order to use only the step-by-step integration method to solve the equation. From the equation (10), mass, stiffness, and speed have a greater influence on the dynamic response of the bridge. For the actual bridge structure, vehicle group distribution length l_p is equal to length of the bridge; therefore, this case is mainly studied as follows.

APPLICATION ANALYSIS

For a reinforced concrete simply supported T-beam bridge with a 30m span, health monitoring is important. The girder width is 12.5m and the height is 1.8m, and the equivalent cross-section moment of inertia is 0.80m^4 . Five servo-control accelerometers and five angle inclinometers were installed in the bridge across and symmetrical position to collect and analyze traffic signals in order to obtain specific models, axle load, and vehicle speed. Fiber Bragg grating strain sensors were installed in the T beam internal and surface for monitoring the bridge force of the important parts.

The numerical analysis method of vehicle-bridge coupling analysis proposed in this paper was used to study the influence of traffic loads to the dynamic characteristics of the bridge. Defining the vehicle group coupling ratio of mass density is the ratio between the coupling uniform mass density and the vehicle group overall mass density. The acceleration responses not considering the coupling mass of the vehicle group and the vehicle group coupled mass ratio of 0.3 is shown in Figure 3. The main body uniform mass density is 30t/m and the vehicle speed is 15m/s. It shows that the bridge dynamic response considering the coupling mass of vehicle group is slightly less but the differences are not obvious. According to experience, a coupled mass ratio can be taken as 0.15 in normal analysis. Considering the influence of the uniform mass of dynamic response of the bridge, the coupling mass ratio is taken as 0.15, and the acceleration and velocity of the bridge are obtained when the mass density is 5t/m and 10t/m, respectively, when the velocity of the vehicle group is 10m/s, as shown in Figure 4. It can be seen that the dynamic response of the bridge increases with the vehicle mass, and the response is like a type of cyclical fluctuation. The maximum amplitude of the acceleration curves when the bridge is generated by a uniform mass density of 5 t/m vehicle group under the different vertical frequency and speed are shown in Figure 4.

The curve families of the maximum amplitude of the displacement when the bridge is generated by the vehicle group with 10 t/m uniform mass density under the different vertical frequency and speed are shown in Figure 5. The corresponding results indicate that the vehicle group and bridge coupling stabilize when the vertical frequency of cars is greater than 0.4Hz. When the speed of vehicle group increases, the displacement and acceleration response of the bridge reveals an increase trend, but the fluctuation change is more obvious. The vehicle-bridge coupling effects cannot be ignored, and the driving speed is an important index for comprehensive analysis.

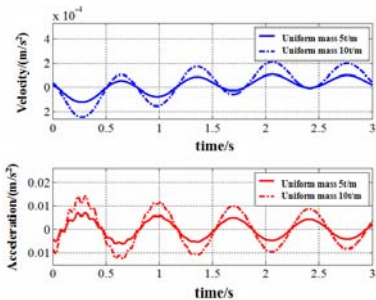


Figure 3. Dynamic responses.

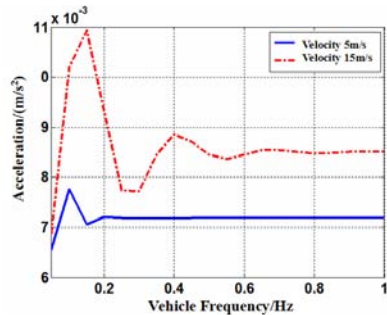


Figure 4. Maximum value with different speed.

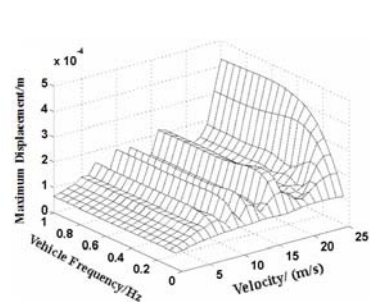


Figure 5. Curves of displacements.

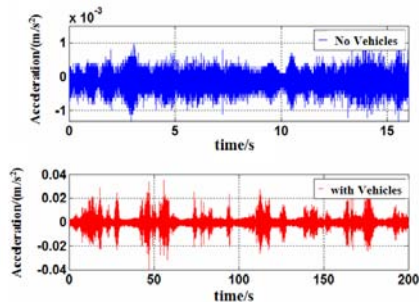


Figure 6. Bridge acceleration signals.

The bridge acceleration signals with or without a vehicle are shown in Figure 6. The total amplitude of measured ambient vibration signals and electromagnetic interference signals always locate in the range of $\pm 1.0 \times 10^{-3} \text{ m/s}^2$. The bridge vibration acceleration caused by vehicle amplitude is generally in the range of $\pm 0.05 \text{ m/s}^2$, coinciding with the results of the numerical analysis. Considering the monitoring requirements, road surface smoothness, and environmental factors, as well as the actual conditions of the initial monitoring results, the monitoring threshold selection of the bridge health monitoring is as follows: When the acceleration amplitude is less than $\pm 5.0 \times 10^{-3} \text{ m/s}^2$ or the displacement amplitude is less than $\pm 1.0 \times 10^{-4} \text{ m}$, the modal analysis function should not be started, otherwise the appropriate data is selected to be analyzed with modal analysis by stochastic subspace method, and the first three

frequencies, the modal shapes and damping ratio is obtained. When the acceleration amplitude is less than $\pm 1.0 \times 10^{-2} \text{ m/s}^2$ or the displacement amplitude is less than $\pm 1.8 \times 10^{-4} \text{ m}$, the modal analysis function should not be started, otherwise the appropriate data is selected for damage detection, determining the possible damage location and degree, and the related data is stored in the long-term pattern. According to the above monitoring strategy, only the important data are recorded and the necessary analysis is carried out, and resources are saved effectively on the premise of guaranteeing the health monitoring effect. For example, if the health monitoring system runs continuously for 24 hours, the storage space in the original database is 504.15 megabyte, but storage is only 57.06 megabyte with the monitoring strategy presented in this paper, and the valid data used for actual analysis coincide with the conclusion better.

CONCLUSION

The effective selection, denoise, analysis, and storage of monitoring data in a bridge health monitoring system has important value in engineering application. The monitoring strategy is proposed by analyzing the signal component and establishing a bridge structure and moving vehicle group dynamic balance equation and analytical expression. The strategy has advantages such as discarding large low SNR or no abnormal data, thus saving the computing resources. By analyzing the data of the actual health monitoring of a girder bridge, the results show that the vehicle load and speed are the main factors influencing the bridge dynamic response. The influence needs to be studied in-depth according to the vehicle bridge coupling dynamics theory and the actual monitoring results. A combination of theoretical analysis and experimental results can determine the monitoring threshold so as to take the appropriate analytical techniques for the data of differing importance. Implementing a health monitoring strategy will improve the cost-benefit ratio of a health monitoring system.

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Estimation and Analysis of Cable Tension Based on Frequency Method

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ABSTRACT

A frequency method was proposed to analyzed cable tension from measured natural frequencies according to cable tension monitoring of cable stayed bridge during construction. Xinzao Pearl River Grand Bridge was chosen as an example. Geometrical calculation simulation model was established to compare measured cable tension to theoretical cable tension during construction. This paper introduces several cable tension test methods. The principle and theoretical derivations under certain conditions, the method of which fundamental vibration frequency is deduced by dominating vibration frequency, the influence of boundary conditions, bending stiffness, and cable sag on the frequency method testing, and the practical application of frequency method were discussed. A spatial finite element model was established based on finite element software MIDAS. The application of forward-analysis iterative method was considered to analyze the cable tension during construction. Then the cable tension test method and analysis principle was explored. Finally, it was shown that measured cable tension based on frequency method was accurate. The measured value is approximately equal to the theoretical value. The experimental error was within the permitted range. The accuracy and reliability of the frequency method was effectively confirmed. The cable tension test for the similar bridges during construction was compatibly applied.

INTRODUCTION

In 1955, the Stromsund Bridge, whose main span is 186.2 meters, was built in Sweden by the German company DEMAG, which marked the birth of the first modern cable-stayed bridge in the word. During the half century, cable-stayed bridge, which was developing rapidly, has become one of the most main bridge types in the world. Cable-stayed bridge construction began in China in the 1970s. China, which has the most number of cable-stayed bridges in the world, has built more than 100 various types of cable-stayed bridge. Cable-stayed bridge is an indeterminate structure with high degrees. The changing of each node coordinate will affect the distribution of structural internal force. In the process of construction, the construction technology is complex and the construction period is long. Internal force and deformation of the structure is influenced by cable sag, temperature, temporary load, creep and shrinkage of concrete, and other complex factors. Cable force affects the internal force and alignment of the cable-stayed bridge directly, which is the most

important criteria to evaluate internal force. The smaller deviation will cause unfavorable bending moment, and then it will have great effect on positive stress of main girder. At present, there are five test methods in common use: pressure gauge method, pressure sensor method, magnetic flex method, fiber Bragg grating vibration method, and frequency method. Frequency method is nearly the only way to measure the cables which have been constructed in China. This work mainly discusses the principle of testing cable force by frequency method. Combining with the example of Xinzao Pearl River Grand Bridge, it established a spatial finite element model for analysis based on finite element software MIDAS. Application of forward-analysis iterative method to analyze the cable tension during construction was considered. Then it illustrated the practical application of frequency method during construction.

PRINCIPLE OF FREQUENCY METHOD

Estimation of cable force of the stay cables is based on string vibration theory. Precision vibration sensor is used to get vibration signal of cable. After filtering, amplifying, and analysis of frequency spectrum, natural frequency of cable is determined by the frequency spectrogram. Then cable force is determined according to the relationship between natural frequency and cable force.

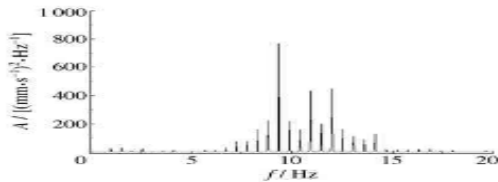


Figure 1. Frequency spectrogram of certain signal

It is supposed that the direction along the cable is the x axis, the y axis is perpendicular to the cable, and any point on the cable vibrate along the y axis. Then the infinitesimal element of cable is analyzed. The infinitesimal in micro-breadth oscillation is subjected to axial force, shear force, as well as bending moment. The force diagram of infinitesimal element of cable is shown as follows:

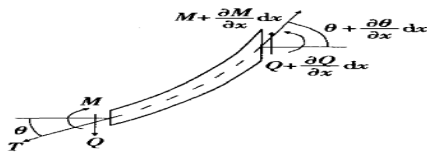


Figure 2. Force diagram of infinitesimal of cable

Considering the bending deformation and neglecting the influence of shear deformation and moment of inertia, the equation of motion at time t is as follows:

$$T \sin \left(\theta + \frac{\partial \theta}{\partial x} \right) + Q + \frac{\partial Q}{\partial x} dx - T \sin \theta - Q = m dx \frac{\partial^2 y}{\partial t^2}$$
 (1)

Neglecting sag-extensibility, the angle θ is small when cable vibrates near the equilibrium state. Eq. (1) is transformed and approximated by

$$T \frac{\partial^2 u}{\partial x^2} + \frac{\partial Q}{\partial x} = m \frac{\partial^2 u}{\partial t^2} \quad (2)$$

Considering the relationship among M , Q , x and y : $\frac{dM}{dx} = Q$, $\frac{\partial^2 y}{\partial x^2} = \frac{M}{EI}$, Eq. (2) can further be normalized by

$$EI \frac{\partial^4 y}{\partial x^4} - T \frac{\partial^2 y}{\partial x^2} + m \frac{\partial^2 y}{\partial t^2} = 0 \quad (3)$$

Where x denotes the coordinate along the cable. The terms y , EI , T , M denote vibration displacement that is perpendicular to the cable at time t , flexural rigidity, tension force, and mass density, respectively.

Suppose that the ends of the cable are hinged, the formula for calculating the internal force of the cable is as follows:

$$T = 4ml^2 \left(\frac{f_n}{n} \right)^2 - \frac{n^2 EI \pi^2}{l^2} \quad (4)$$

Where

$$f_n = n \sqrt{\frac{T}{4ml^2} + \frac{n^2 EI \pi^2}{4ml^4}} \quad (5)$$

Where n denotes the order of natural frequency. The terms f_n , l denote the n th natural frequency in Hz, length of cable, respectively.

According to Eq. (5), the spectrogram is spectrum line whose spacing gradually increases. The slenderness ratio of cable is large, and the ratio of bending stiffness of cable to the fourth power of the length of cable is small. Under the condition of small order, the second term is much smaller than the first one in the square root. So the second one can be neglected. Eq. (5) is simplified to

$$f_n = n \sqrt{\frac{T}{4ml^2}} = n f_1 \quad (6)$$

Where f_1 denotes the first natural frequency, which is the fundamental frequency.

Then the spectrogram is the spectrum line whose spacing is equal. The formula of calculating the internal force of the cable can be written by

$$T = K f_1^2 \quad (7)$$

Where K is proportion coefficient expressed as

$$K = 4ml^2 \quad (8)$$

The vibration signal of cable, which is composed of the signal of polyharmonic vibration, is compound vibration signal. There will be multiple peaks on spectrogram after spectrum analysis, each peak stands for one natural frequency. According to the Eq. (6), the natural frequency of cable approximates integral multiple of fundamental frequency. When the fundamental frequency is measured in use of cable force measured instrument, it shows the frequency of the highest peak. It is the main vibration frequency. So, in order to get the fundamental frequency, the order n should be calculated accurately. And if all the natural frequency completely appears on the spectrogram, order n is the order of main vibration frequency. The distance of every two adjacent natural frequency is equal. But actually the vibration signal of some order is exceedingly weak when the cable vibrates. So, it cannot be seen on the spectrogram so that the distance between two peaks is not equal. Thus the distance between the two peaks whose distance is the shortest is taken as the fundamental frequency, and the real number of natural frequency is deduced. Then

the value of n is calculated. If the main vibration frequency and its order are known, the fundamental frequency can be calculated by Eq. (6). Then the cable force is calculated by Eq. (8).

ANALYSIS OF INFLUENCE FACTORS THAT AFFECT ESTIMATING CABLE FORCE

Effect of Boundary Condition

Because there are anchor devices at the ends of the cable, the boundary condition of the cable is between hinged and clamped. The actual situation is closer to the clamped support. Without considering sag-extensibility, the values of the cable force which were calculated by each boundary condition are equal when the bending stiffness is neglected. Eq. (4) showed that the flexural rigidity is negligible when the cable is exceptionally long. The value of cable force which is calculated without considering flexural rigidity is larger. But it does not exceed 5% of the value which is calculated by considering the flexural rigidity generally. However, for the short cable, the value will be more than 5%. At this point, cable force should be modified by choosing a reasonable effective length of cable. Because the stiffness of the anchor head of both ends and corresponding connection part is larger than the stiffness of other parts, according to analysis and measured results, it can be found that taking the difference between the distance between each anchor point at the ends of the cable and the length of connecting cylinder as the effective length of the cable can meet the requirement of the accuracy of estimating cable force on construction when the cable force is calculated by Eq. (7).

Effect of Flexural Rigidity

Considering flexural rigidity, the formula of calculating the internal force of the cable can be conveniently written by

$$T = 4ml^2 \left(\frac{f_n}{n} \right)^2 - n^2 D \quad (9)$$

Where D is a coefficient expressed as

$$D = \frac{\pi^2 EI}{l^2} \quad (10)$$

Then

$$f_n = n \sqrt{\frac{T + n^2 D}{4ml^2}} \quad (11)$$

For the cable of same length and diameter, D is a constant. At this point, f_n / n is no longer a constant, but it is monotonically increasing with the increase of n . Reflected on the spectrogram, the distance between two adjacent peaks increase with the increase of order n . For the short cable, the error of the cable force which is calculated by Eq. (7) is relatively big. So, it does not meet the requirement of the accuracy. In this case, in order to reduce the error, cable force should be calculated by low-order frequency instead of the fundamental frequency after calculating the frequency of cable.

Effect of Sag-Extensibility

Generally speaking, the sag of cable appears under self-weight, thus the line style of the cable is catenary. The elongation of cable consists of the elongation of

elasticity and overcoming the sag when the cable is stretched. For convenient calculation, the method of equivalent elastic modulus is adopted. The effect of sag is considered into the formula of elastic elongation.

When the influence of sag on cable force is considered into long-span cable-stayed bridge, the equivalent elastic modulus can be calculated by the Ernst formula.

As shown in Fig. 3, it was assumed that the self-weight of cable is uniform load whose intensity is q , and the deflection in the middle of the cable is f_m .

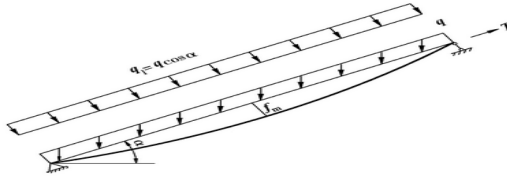


Figure 3. Force diagram of stay cable

Because the cable does not bear bending moment, take moments about the midpoint m of the cable by using the following equation:

$$T \cdot f_m - \frac{1}{8} q l^2 = T \cdot f_m - \frac{1}{8} q \cos \alpha \cdot l^2 = 0 \quad (12)$$

Where

$$f_m = \frac{q l^2}{8 T} \cdot \cos \alpha \quad (13)$$

Where T denotes cable force. The terms l , α denote the length of cable and the angle between cable and horizontal line respectively.

The line style of the cable is catenary under the effect of the self-weight of the cable. Because the value of f_m is relatively small, the line style is considered as parabola. The length of cable can be approximated by

$$S = l + \frac{8}{3} \cdot \frac{f_m^2}{l} \quad (14)$$

According to Eq. (13) and Eq. (14):

$$\Delta l = S - l = \frac{8}{3} \cdot \frac{f_m^2}{l} = \frac{q^2 l^3}{24 T^2} \cos^2 \alpha \quad (15)$$

Then

$$\frac{d \Delta l}{d T} = - \frac{q^2 l^3}{12 T^3} \cos^2 \alpha \quad (16)$$

As shown in Fig. (4). When equivalent elastic modulus is calculated by Ernst formula, The elastic modulus, which represents the effect of sag, is tangent modulus of elasticity by using the following equation:

$$E_{f_1} = \frac{d \sigma}{d \varepsilon_f} \quad (17)$$

According to $\sigma = \frac{T}{A}$, $\varepsilon_f = \frac{\Delta l}{l}$, the tangent modulus of elasticity E_{f_1} can be transformed by

$$E_{f_1} = \frac{d \sigma}{d \varepsilon_f} = \frac{d \frac{T}{A}}{d \frac{\Delta l}{l}} = \frac{dT}{d \Delta l} \cdot \frac{l}{A} \quad (18)$$

Where E_{f_1} denotes tangent modulus of elastic which represents the effect of sag. σ denotes tensile stress of cable. And ε_f denotes tensile strain caused by the effect of sag.

Plug Eq. (16) into Eq. (18)

$$E_{f_1} = \frac{12T^3}{q^2 l^3 \cdot \cos^2 \alpha} \cdot \frac{l}{A} = \frac{12T^3}{q^2 l^2 A \cos^2 \alpha} \quad (19)$$

According to $q = \gamma \cdot A$, $L = l \cdot \cos \alpha$, $\sigma = \frac{T}{A}$

$$E_{f_1} = \frac{12T^3}{(\gamma A)^2 L^2 A} = \frac{12\sigma^3}{\gamma^2 L^2} \quad (20)$$

Where L denotes horizontal projection length of cable

Then the equivalent elastic modulus E_{eq_1} can be described by

$$E_{eq_1} = \frac{\sigma}{\varepsilon_e + \varepsilon_{f_1}} = \frac{\sigma}{\frac{\sigma}{E_e} + \frac{\sigma}{E_{f_1}}} = \frac{E_e}{1 + \frac{E_e}{E_{f_1}}} \quad (21)$$

Plugging Eq. (20) to Eq. (21), Eq. (21) can further be normalized by

$$E_{eq_1} = \frac{E_e}{1 + \frac{\gamma^2 L^2 E_e}{12\sigma^3}} \quad (22)$$

Where E_{eq_1} denotes the equivalent elastic modulus which is calculated by Ernst formula.

The terms E_e, ε_e denote the elastic modulus of cable and the tensile strain of cable respectively.

However, when the sag of cable is considered in ASCE, the elastic modulus, which represents the effect of sag, is secant modulus shown in the Fig. (4), and is defined by

$$E_{f_2} = \frac{\sigma_1 - \sigma_0}{\varepsilon_1 - \varepsilon_0} \quad (23)$$

Where E_{f_2} denotes secant modulus of elastic which represents the effect of sag

According to $q = \gamma \cdot A$, $L = l \cdot \cos \alpha$, $\sigma = \frac{T}{A}$

$$E_{f_2} = \frac{24\sigma_1^2 \sigma_0^2}{\gamma^2 L^2 (\sigma_1 + \sigma_0)} \quad (24)$$

Plugging Eq. (22) to Eq. (19), the equivalent elastic modulus can be written by

$$E_{eq_2} = \frac{E_e}{1 + \frac{\gamma^2 L^2 (\sigma_1 + \sigma_0) E_e}{24\sigma_1^2 \sigma_0^2}} \quad (25)$$

Where E_{eq_2} denotes the equivalent elastic modulus which is calculated according to the rule of ASCE.

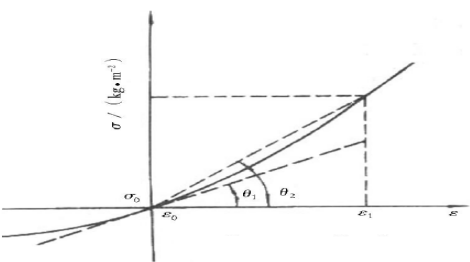


Figure 4. Tangent and secant elasticity modulus

THE ANALYSIS FOR INSTANCE

Engineering Background

Xinzao Pearl River Bridge is a long span bridge on the express way from Xinzhou to Hualong in Guangzhou province. The bridge is located in the north of the channel of Xinzao. The area is delta plain and local residual hill, whose terrain is relatively flat.

Xinzao Pearl River Bridge is a prestressed concrete cable-stayed bridge with double pylons and single cable plane, whose tower, girder and pier are consolidated together. Spans are attributed as (64+140+350+140+64) m, each plane has twenty-six couples of stay cables which use Semi parallel wire strands, and each of the cables is composed of 187 to 283 pieces of steel wire. The longest cable approximate 190m. At each side span, Auxiliary piers, which are conducive to enhancing wind resistance on construction and strengthening the rigidity of structure on operation stage, are set up at 140 meters away from the center line of main tower. The elevation of Xinzao Pearl River Bridge is shown in Fig. 5.

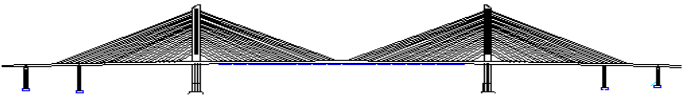


Figure 5. Elevation of Xinzao Pearl River Grand Bridge

Simulation Model

Emulating calculation of construction control of cable-stayed bridge is that the forced status of completed bridge and construction is simulated and analyzed by establishing a reasonable finite element model and applying appropriate analysis methods.

The spatial finite element model of Xinzao Pearl River Bridge was built by using finite element software MIDAS. The full bridge was divided into 689 nodes and 574 elements, which includes 462 beam elements and 112 truss elements. The computation model of the structure is shown in Fig. 6.

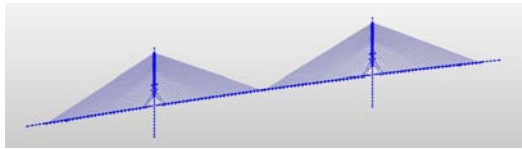


Figure 6. The model of Xinzao Pearl River Grand Bridge

The rational finished bridge state is decided by utilizing the function of unknown load factor in Midas/Civil during the process of emulating calculation. The finished bridge state mainly includes alignment state and internal force of the structure after completing the bridge. The reasonable alignment state can be determined by designing pre-camber in laying elevation at each stage in the process of the construction. And the reasonable internal force state is decided by cable force. So, the rational finished bridge state is mainly determined by the reasonable final cable force. At the phase of completed bridge, the calculation procedure of final cable force is shown as follows:

Establish the model of completed bridge → decide the load cases of main girder and cable → input the load and initial tension of cable → establish load combination of dead load and cable load → set the controlled condition and solve the unknown load coefficient → inspect analysis results and output the final cable force.

After determining the reasonable final force, the construction process and method and the cable force of reaching target state was decided by considering the completed bridge state as the target state. The reasonable forced status during construction was decided.

According to the construction stages and construction load, the cable tension force on construction was calculated by forward-analysis iterative method. First, assuming that the self-weight of corresponding girder segment plus secondary loads is equivalent to cable force, the initial tension was determined preliminarily. The final cable force was calculated. Then the calculated final cable force was subtracted by the target final cable force, superposing the difference on the calculated initial cable force, and considering the result as new initial cable force. The calculation did not finish until the calculated final cable fore was equal to the target final cable force. Then the cable tension force on construction was determined.

Contrastive Analysis of Measured Cable Force and Cable Force Calculated by Numerical Simulation

Because the sag of cable after the first tension and the second tension is larger than that after the third tension, the measured cable force after the first and the second tension, which was calculated by Eq. (7), is not accurate. When the cable is stretched for the third time, the control is based on cable force, otherwise, the control is based on elevation. We chose the calculated results of simulation analysis of cable 1, pier 22 to compare to the measured results.

The theoretical cable force of cable 1, pier 22 which was determined by simulation analysis and the theoretical fundamental frequency which was calculated by Eq. (7) are shown in Table 1.

Table 1. Theoretical stay cable fundamental frequency

serial number of cable	Length of stay cable (m)	mass density of cable	proportion coefficient	Theoretical cable force	Theoretical fundamental frequency
B01	26.291	56.495	156.204	3843	4.960
Z01	25.912	56.495	151.729	3746	4.969

When cable force is measured, stay cable should be tensioned hierarchically. The cable tension force is controlled by tensioning jack and fuel meter. Fundamental frequency of stay cable, which is measured by cable force measured instrument, is determined by the average value of triple measurement. Then the field measured values are compared with the theoretical values. The requirements of design allow it to cause differences in values by no more than 2.5 percent and 50 KN.

The comparison between the measured cable force of cable 1 after the third tension and the theoretical cable force is shown in Table 2. It was shown that the results for the cable force monitoring of cable 1 after the third tension was satisfied according to the frequency method and the formula $T = Kf_1^2$. The cable force can be determined accurately by frequency method when stay cable is under the effect of load. The field measured values approximated the tension cable force during construction which was calculated by simulation model and satisfied the requirements of engineering application because the errors are very small, which are within the allowed range of design requirements.

Table 2. The comparison between measured cable tension and theoretical cable tension

Serial number of cable	Location of cable	Measured fundamental frequency (Hz)		Measured cable force (KN)	Theoretical cable force (KN)	Difference of cable force	Ratio of difference of cable force and theoretical cable force
		Measured value	Average value				
B01	upstream	4.970					
		4.962	4.963	3848	3843	5	0.13%
		4.958					
	downstream	4.989					
		4.971	4.983	3879	3843	36	0.94%
Z01	upstream	4.990					
		4.977					
		4.960	4.975	3755	3746	9	0.24%
	downstream	4.989					
		4.966					
		4.971	4.968	3745	3746	-1	-0.03%
		4.967					

CONCLUSIONS

The objective of this work is to introduce an approach to estimate the cable tension forces from the measured frequencies. Main conclusions are as follows: Frequency method is nearly the only way to measure the cables which have been constructed in China. When the cable is relatively long, the effect of flexural rigidity on fundamental frequency can be neglected. Otherwise, the flexural rigidity should be considered into the calculation. The deduction of the relationship between cable force and fundamental frequency is under the assumption that the sag of cable is neglected, otherwise, the results calculated are not accurate. The calculated cable forces based on frequency method are relatively precise under normal circumstance. The method is appropriate for cable force measurement during construction.

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Investigation on Shear Lag Effects of Extradosed Cable-Stayed Curved Bridge under the Influence of Curvature Radius

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ABSTRACT

As a new type of bridge, extradosed cable-stayed curved bridge is widely used in mountainous area. The box girder bears not only axial pressure but also a considerable part of moment and shear force. The existence of curvature makes the force state more complex and the spatial stress state uneven. Moreover, the broadening of box girder leads to serious shear lag effect. Based on the engineering background of Longjing River Bridge, the positive stress distribution and shear lag effect in top and bottom slab of the curved box girder under different curvature radius during the completion phase were studied using FEA method. The research indicated that the difference of shear lag effect between the inside and the outside of the curved box girder would be more serious if the curvature radius decreases under dead load action. The positive and negative shear lag effect would simultaneously appear in inner and outer side of flange plate in one section under the influence of curvature radius. Curvature radius has a greater influence on the top plate than the bottom plate. The shear lag effect coefficient would be calculated as straight bridge when the curvature radius is greater than 800m.

INTRODUCTION

As a new type of bridge, extradosed cable-stayed curved bridge is widely used in the highway construction of mountainous areas. The box girder bears not only axial pressure but also a considerable part of moment and shear force. The existence of curvature makes the force state more complex and the spatial stress state uneven. Now both the bridge span and the girder width of the cable-stayed bridge built at home and abroad keep increasing. As the width-span ratio and width-height ratio of the girder increase, the shear lag effect increases. Since there are no clear rules and quantitative basis, the shear lag effect by the influence of curvature radius on wide flange plat box beam of the extradosed cable-stayed curved bridge is also no basis (Evans&Ahmad1993, JTG D60-2004 2004, Zhang&Li2008, Zhi2011). And the existed analytic methods of shear lag effect calculation on curved box girder are cockamamie for engineer and hardly to be applied (Huang 1994). Due to these reasons, the shear lag effect was always ignored, which leads to transverse crack in box girder.

SHEAR LAG COEFFICIENT OF CURVED BOX GIRDER

For better describing the shear lag effect of curved box girder, a shear lag coefficient λ is introduced. Considering the bending-torsion coupling effect, the

generalized shear lag coefficient λ is defined as: $\lambda = \text{Normal stress of each point of the section} / \text{Mean normal stress of the section}$ (Luo 2005)

Since there is no consensus on the definition of the shear lag coefficient in practical application, λ is calculated as follows in this article: firstly, the integral value of the normal stress along the width of top or bottom slab are divided by the width of corresponding slab in order to obtain a normal stress similar to the one calculated by general beam theory. Then the actual stress value of each point is divided by the average value to obtain the shear lag coefficient.

ESTABLISHMENT OF 3D FEA MODEL

Based on the engineering background of Longjing River Bridge, the shear lag effect of bridge girder under cable force is studied. It is an extradosed cable-stayed curved bridge with single cable plane and two cable tower, while the designed diameter is 850m. The bridge structure is pier-tower-beam fixed system and has three spans in all (86m+160m+86m). The width of bridge floor is 28m with 4° transverse slope. The three cells single box girder of the bridge with variable cross section, the girder is 6.5m deep at the top of pier and 3.0m in mid-span. The rectangle pier with double thin-wall is 60m high and the pylon use solid rectangle section with a height of 28m. The six pairs of parallel stayed-cable are arranged on both sides of the cable tower, the space between cables is 1m on the cable tower and 4m on the girder. According to the design specification, the material of the main beam of model is C55, the unit weight of which takes 25KN/m³, elastic modulus E value of which is 3.55×10⁴MPa, Poisson's ratio of which takes 0.2. The elastic modulus value of prestressed steel wires is 1.95×10⁵MPa and the tension stress is 3750KN. For the concrete girder bridge, the proportion of dead load is great, so only the state of weight + cable force is analyzed in this paper.

In order to know the shear lag effect of box girder under the curvature radius, the models of winding bridge with the curvature radius of 250m、450m、650m、860m and straight bridge are established. According to the “Technical Standard of Highway Engineering” (JTG B01-2003, 2003), the minimum radius of highway circular curved is 250m, so the curvature radius above is selected. On the basis of references, the mean normal stress of each section almost changed little, while the maximum shear lag coefficient varied with the variation of curvature radius and only some certain parts (middle of side-span, start element, 1/4 of mid-span etc.) changed drastically. Due to limited space, all the data obtained from this 4 control sections: the middle section of side-span, beginning section of start element, 1/4 section and 1/2 section of mid-span. The force and shear lag coefficient of control sections are analyzed under the different condition during the finished stage. For better to describe, several points are set in top and bottom slab. The width of top slab is 28m, while that of bottom slab changes, so 29 points of top slab at each control section, 19 points of bottom slab at 1/4 section of mid-span and middle sections of side-span and mid-span, 14 points of segment 0# bottom slab were analyzed as needed. The layout of analysis points is shown in Figure 1.

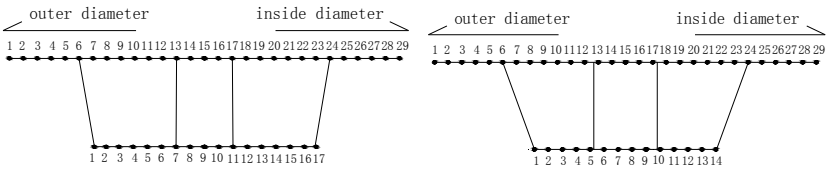


Figure1. The layout of analysis points

The 3D solid model is established and the simulation analysis is also carried out with FEA software MIDAS/Fea3.0. Solid element is used as the box girder, cable tower and piers, while the tension-only element is used as cables. Taking the advantage of symmetry, the semi-structure of the bridge is established with proper constraints, and then the FEA model is formed by automatic meshing method. The FEA model is showed as figure1. The box girder, cable tower and piers are fixed together, while a bilateral and an uniaxial bearing are applied in the two side of the main beam, and the bottom of the piers are also be fixed. The FEA model of the whole bridge and standard segment are separately shown in Figures2 and 3.



Figure 2.The FEA model of bridge

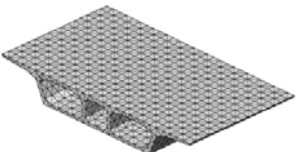
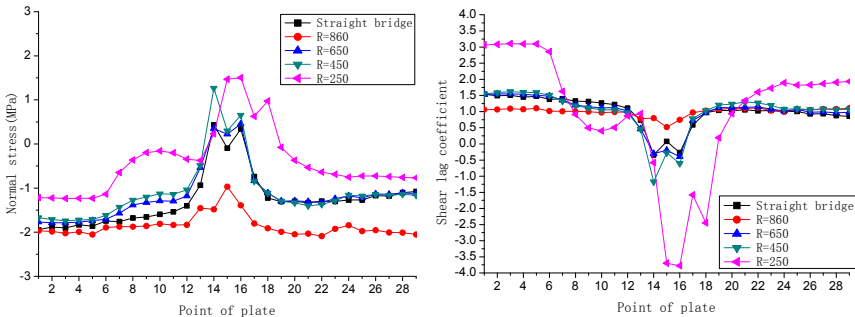


Figure 3.The FEA model of standard segment

COMPARISON OF CALCULATED RESULTS AND ANALYSIS

Figures 4~11 show the comparison of longitudinal normal stress and shear lag coefficient of several typical sections under different load conditions.



a) transverse distribution of stress

b) transverse distribution of shear lag coefficient

Figure 4. The shear lag coefficient of top slab in middle section of side-span

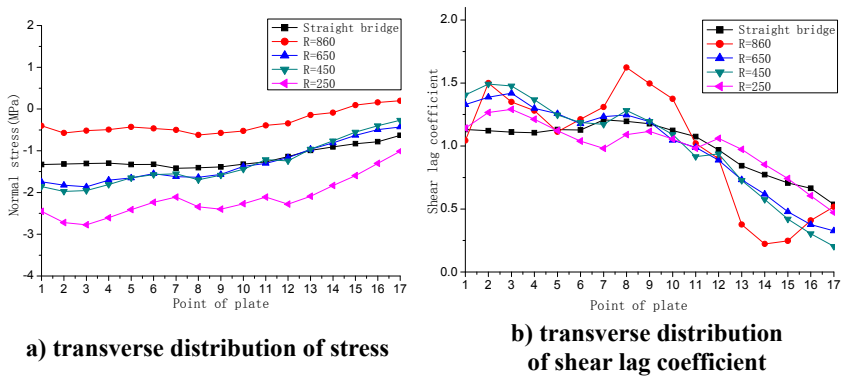


Figure 5. The shear lag coefficient of bottom slab in middle section of side-span

As shown in Figure 4, with decreasing curvature radius, pressure stress of majority control points of the top slab in 1/2 section of mid-span decreases gradually, and the stress distribution is more uneven when the curvature radius is 250m. The stress of No.13 to 16 points mutate to tensile stress under effect of cable force due to that these points are close to the cable anchor point. The shear lag coefficient of outer diameter is greater than that of inner diameter, and the coefficient of these control points close to the rib plate has great changes, the shear lag effect becomes much more significant when curvature radius is 250m and 450m. Figure 5 shows that pressure stress of bottom slab increase with decreasing curvature radius. The shear lag coefficient of outer diameter is greater than that of inner diameter. The outer diameter side appears positive shear lag effect, while the opposite appears negative. Overall, curvature radius has a slight effect on shear lag effect of bottom slab. It can be concluded that the shear lag effect of this section becomes significant with decreasing curvature radius.

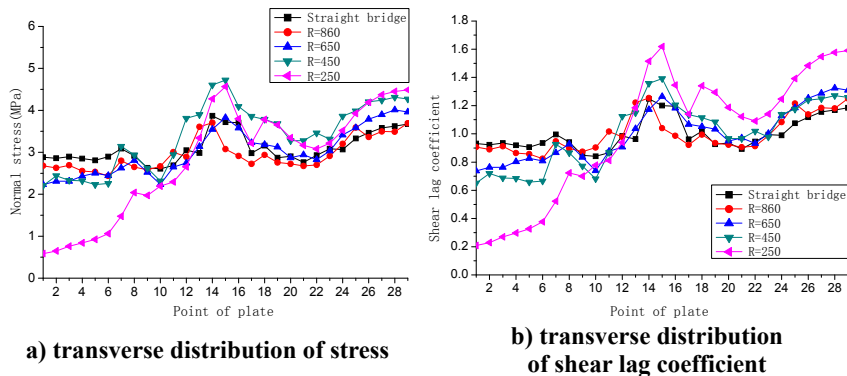


Figure 6. The shear lag coefficient of top slab in beginning section of start element

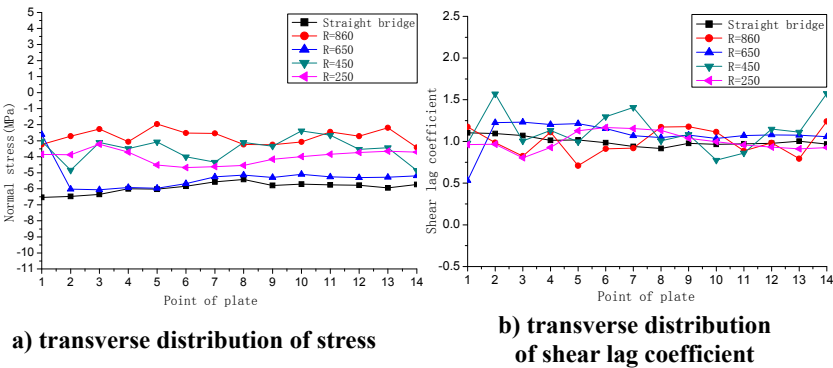


Figure 7. The shear lag coefficient of bottom slab in beginning section of start element

As shown in Figure 6, the segment 0# top slab appears tensile stress state and the outer diameter side appears negative shear lag effect, while the opposite side appears positive. The coefficient of these control points close to the rib plate has great changes. The stress distribution is more uneven while the curvature radius decreases. As shown in Figure 7, the segment 0# bottom slab appears press stress state and curvature radius has a slight effect on shear lag effect of bottom slab.

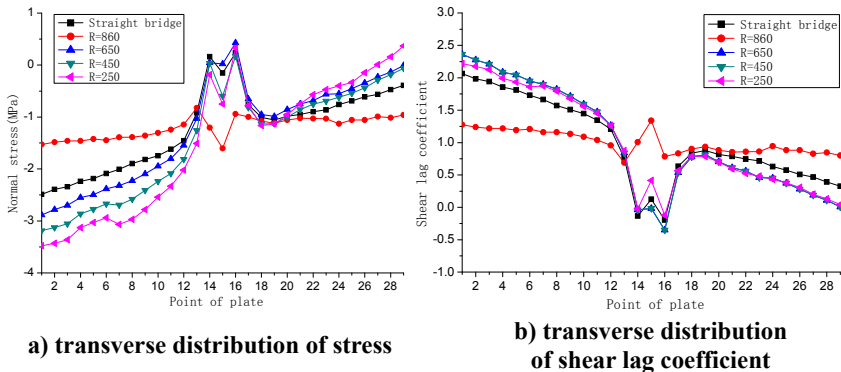


Figure 8. The shear lag coefficient of top slab in 1/4 section of mid-span

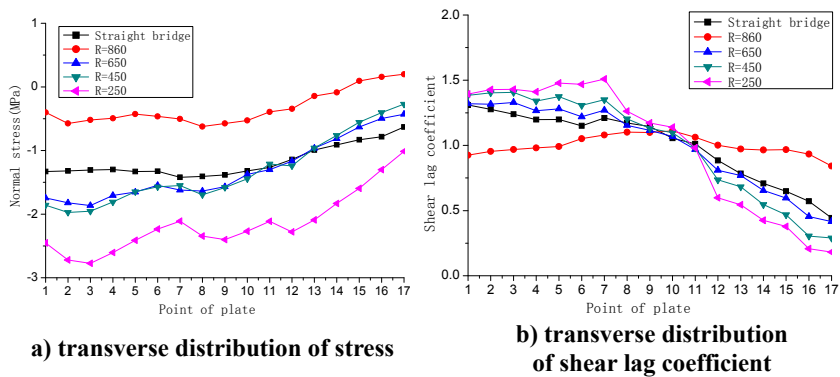


Figure 9. The shear lag coefficient of bottom slab in 1/4 section of mid-span

As shown in Figure 8, with decreasing curvature radius, pressure stress of majority control points of the top slab in 1/4 section of mid-span decreases gradually, and the stress distribution is more uneven, stress difference between the outer diameter side and the inner diameter side is larger when the curvature radius is 250m. The stress of No.13 to 16 points mutate to tensile stress under effect of cable force due to that these points are close to the cable anchor point. The shear lag coefficient of outer diameter is greater than that of inner diameter, and the coefficient of these control points close to the rib plate has great changes. The outer diameter side appears positive shear lag effect, while the opposite side appears negative. Figure 9 shows that pressure stress of bottom slab increase with decreasing curvature radius. Basically the outer diameter side appears positive shear lag effect, while the opposite side appears negative, and curvature radius has a slight effect on shear lag effect of bottom slab. It can also be concluded that the shear lag effect of this section becomes significant with decreasing curvature radius.

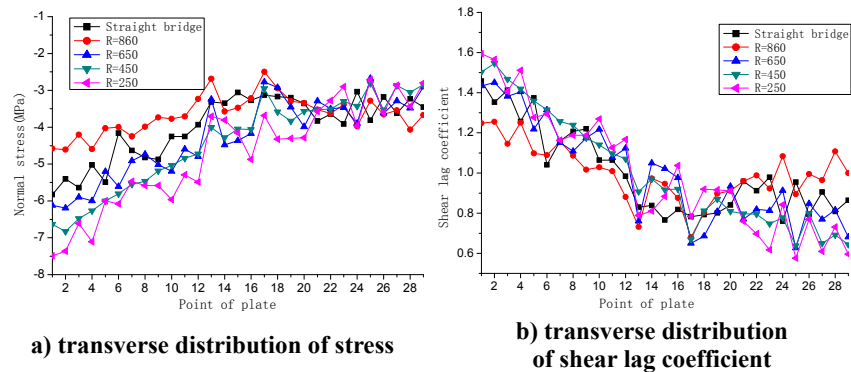


Figure 10. The shear lag coefficient of top slab in middle section of mid-span

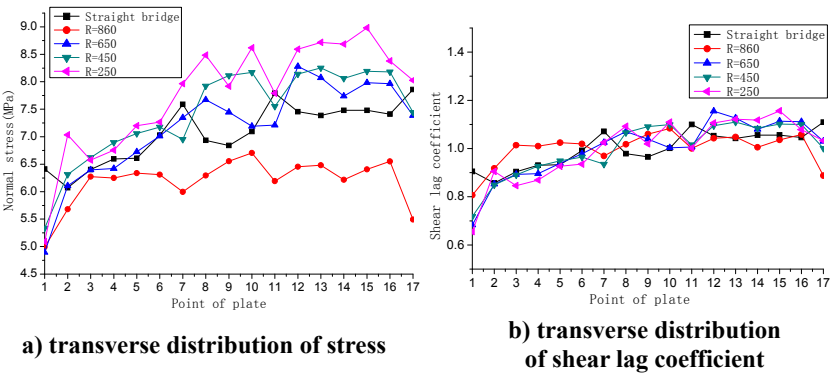


Figure 11. The shear lag coefficient of bottom slab in middle section of mid-span

As shown in Figure10, with decreasing curvature radius, pressure stress of the top slab in middle section of mid-span increases. The outer diameter side appears positive shear lag effect, and the inner side appears negative shear lag effect. The coefficient of these control points close rib plate shows a significant change. Figure 11 shows that the tensile stress of bottom slab increase with increasing curvature radius, and the curvature radius has a slight effect on the shear lag coefficient. In general, the shear lag effect of middle section of mid-span becomes significant with decreasing curvature radius.

CONCLUSIONS

Based on the engineering background of Longjing Bridge, the stress distribution and shear lag effect of bridge girder under different curvature radius were analyzed through FEA simulation in this article. Conclusions can be summarized as below:

Compared to the outer and the inner diameter sides of straight bridge, which presents symmetrical effect under action of self-weight, the shear lag effect of curved bridge is much more uneven. The difference between the outer and the inner sides increases with the decrease of curvature radius. Curvature radius has a significant impact on shear lag effect of curved box girder. Except for the top slab of 0# segment and the bottom slab of middle section of mid-span, the shear lag coefficients of other sections on the outer diameter side were larger than that on the inner side. In most cases, shear lag coefficient of section center basically remain unchanged with curvature radius variation. Curvature radius has great effect on the shear lag of the top slab, while it affects slightly on that of the bottom slab.

Both positive and negative shear lag effect appeared at the same section under different curvature radius. Due to the complicated stress distribution, it cannot describe the shear lag effect accurately with positive or negative shear lag states and maximum shear lag coefficient. The maximum stress appeared at the cantilever end flange in some sections of top slab, namely the shear lag coefficient is maximum, which should pay special attention in box girder design.

According to the results, shear lag coefficient errors of top and bottom slab

are not calculated as straight bridge if the curvature radius is larger than 800m. But when curvature radius is 250m, the maximum error of the top slab reaches 28% and that of the bottom slab reaches 15% if it is calculated as straight bridge by ignoring the influence of curvature.

There exists significant shear lag effect near concentrated load position such as the cable force. Therefore, these positions should be paid high attention to during the construction process. The shear lag effect should be fully considered in box girder design. It should consider setting up a safety factor accounting for shear lag effect when doing the overall design with only plane truss analysis program. The safety factor value needs further study to be determined. Otherwise the spatial finite element analysis should be carried out to ensure the structure safety.

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Creeping Mechanism Analysis of Curved Bridges

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ABSTRACT

Creeping diseases can reduce driving comfort and endanger structural safety, so it is necessary to analyze the causes of creeping. Based on existing research, the factors of the creeping phenomenon using TianNingSi overpass as an example related to the features of radial displacement under dead load, live load, concrete shrinkage and creep, temperature, prestress, etc., were analyzed. The study shows that centrifugal force is the main cause of creeping in curved bridges, which have heavy traffic and large curvature radii. The deficiencies in infrastructure and bearing form should be the main cause of creeping disease. Bearing form optimization can prevent creeping diseases.

Key Words: Creeping phenomenon; centrifugal force; bearing eccentricity

INTRODUCTION

With the rapid development of bridge construction in China, curved bridges are more and more widely used in cities. Curved bridges for a variety of complex terrains have good adaptability, especially in urban grade separation bridges. They not only can improve road capacity, but also can beautify the landscape in a city. However, in recent years, a variety of diseases have appeared around some previously-constructed bridges in China, ranging from aesthetic impacts to reduced driving comfort, and causing serious public safety incidents such as collapsed bridges. The “creeping” phenomenon has attracted wide attention in civil engineering.

Because the force mechanism in curved bridges is very different from straight bridges, creeping diseases can be easily produced in curved bridges. Early designers did not understand the curved bridge structure calculation theory. With a limited calculation level, the traditional method was used with the straight-girder model, which now has been revised based on experience. The data obtained, however, are often inaccurate. As a result, creeping and other diseases have shown widespread emergence in curved bridges. With the constant improvement of computation theory, curved bridge force and deformation characteristics can be better understood. Reasonable proposals can be put forward in the design stage, and bridge diseases can be prevented.

MODELING

Bridge data. This paper uses TianNingSi overpass as a modeling example for analysis. This bridge was built in 1991 and is an eight-span, prestressed concrete, continuous box girder bridge. The total span is 217.098m. An illustration of the bridge is shown in Figure 1.

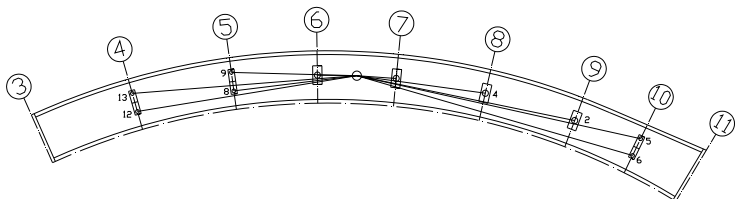


Figure 1. TianNingSi overpass bridge.

In 2011, TianNingSi overpass experienced the “creeping” phenomenon. Some spherical bearing radial displacement on a single pier was transfinite. In 2012, special bearing detection creeping value was detected on this bridge. The test results are shown in Figure 2.

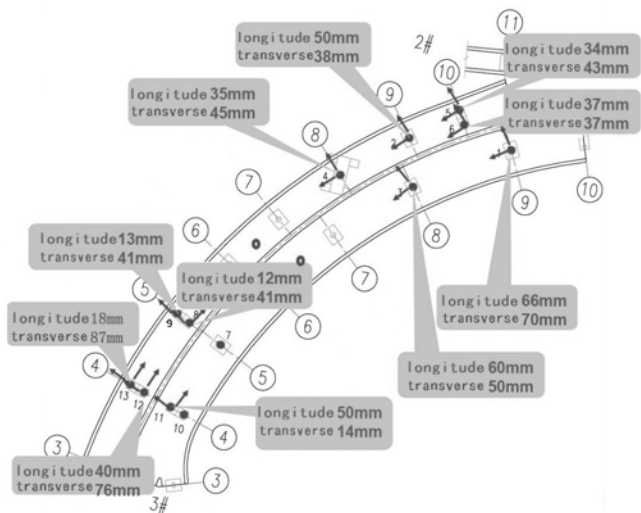


Figure 2. Displacement of bridge.

Modeling in grillage method. This bridge was modeled with MIDAS/CIVIL software and the grillage method. The modeling process was in accordance with JTG D60-2004, JTG D62-2004 standards. Dead load, prestressed, temperature, live load, centrifugal force, shrinkage, and creep load were loaded, and the bridge construction stage was divided. The final model is shown in Figure 3.

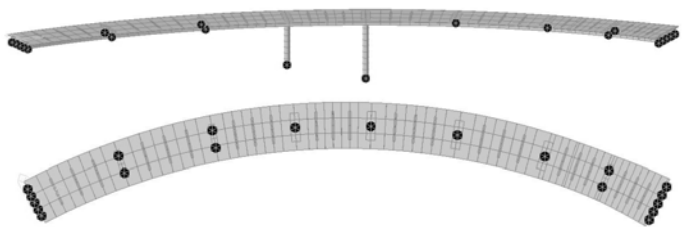


Figure 3. Final bridge model.

ANALYSIS

Itemized analysis. The generation of creeping diseases is a very complicated process and is caused by many reasons, such as geometric eccentricity, temperature load, prestressed effect, vehicle loads, concrete shrinkage, and creep. Moreover, these factors also affect each other. Therefore, it is necessary to study how creeping diseases emerge when caused by each element. Reasonable proposals to reduce creeping value can be put forward.

Geometric eccentricity. Different from a straight girder bridge, a curved bridge’s center of gravity is relative shear center to the outside, and cross slope has been set on a curved bridge. As a result, the eccentric tendency increases. Therefore, in comparison with a straight girder bridge, a curved bridge has a creeping tendency.

For the TianNingSi overpass, creeping produced by dead load is minimal. This results mainly because the central angle in this bridge is smaller than in any other curved bridge, so the eccentric tendency is negligible. If the curved bridges have small traffic and a large central angle, the result should be different.

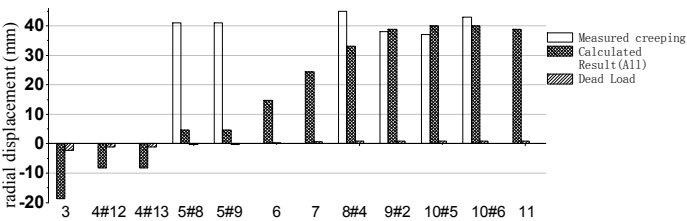


Figure 4. Creeping caused by geometric eccentricity.

Temperature load. Temperature actions include integral temperature rise and fall and the vertical temperature difference between the top flange and the bottom flange. Because of the difference between the inside and outside arc length in a curved bridge, the different deformations between the inside and outside arc could be easily led by the integral temperature rise and fall process. The creeping phenomenon can be easily stimulated by this process. It was also found that the vertical temperature difference between the top flange and the bottom flange is the main reason that results in the torsion of a curved bridge, which can also lead to the creeping phenomenon. Temperature load is a significant factor that cannot be ignored.

For the TianNingSi overpass, creeping produced by temperature effects is minimal. This results mainly because the curvature radius is smaller than any other curved bridge, so the difference between the inside and outside arc length is negligible. If curved bridges have a small curvature radius, the result should be different.

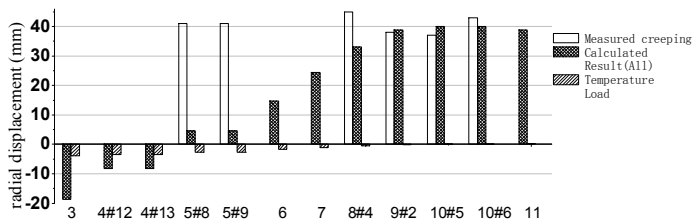


Figure 5. Creeping caused by temperature load

Concrete shrinkage and creep. For a curved bridge, displacement caused by concrete shrinkage in a plane belongs to the nature of the arc’s expanding or contracting. In this process, the arc radius changes, but central angle does not. As a result, deformation of the outer arc has a larger deformation than the inner arc. Therefore, when the shrinkage of concrete occurs, curved girder bridge displacement occurs in two directions. One of these is the radial direction. As time passes, the creeping volume increases. The theory is basically same as the temperature effect.

For the TianNingSi overpass, creeping produced by concrete shrinkage and creep are more obvious. This results mainly because the total span in this bridge is longer than in any other curved bridge, so the shrinkage and creep phenomenon in this bridge are obvious. As a result, creeping caused by concrete shrinkage and creep cannot be ignored.

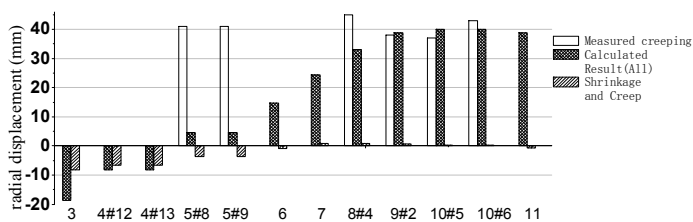


Figure 6. Creeping caused by concrete shrinkage and creep.

Prestress effect. In curved bridges, prestress tendons should first meet the longitudinal moment requirements. As a result, the prestress tendons’ length in a positive moment section is much larger than the length in a negative moment section, so the prestress tendons located under the beam are much longer than the tendons on the top of beam. Finally, the total torque direction generated by prestress is outward. Therefore, prestress can produce the creeping phenomenon.

For the TianNingSi overpass, creeping produced by prestress is not obvious. This results mainly because the curvature radius in this bridge is smaller than in any other curved bridge. Therefore, prestress is not the main reason for creeping diseases.

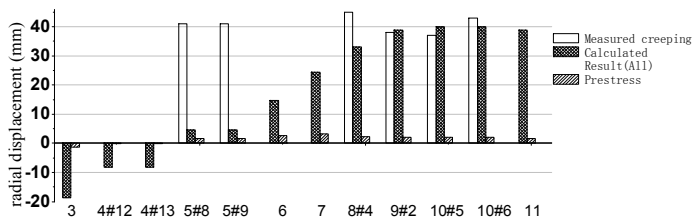


Figure 7. Creeping caused by prestress effect.

Vehicle load. As vehicles travel on a curved bridge, the beam will be subjected to vertical force (vehicle gravity, impact force), tangential force (braking force), and transverse force (centrifugal force). For vertical force, the vehicle eccentric can generate torsion. Transverse force is continuously loading over a very long period of time. In addition, when the creeping phenomenon emerged, because of bearing friction and other factors, lateral displacement cannot be fully recovered after the external loads disappear. A few years later, creeping diseases will occur.

For the TianNingSi overpass, creeping produced by centrifugal forces are more obvious than other loads for a variety of reasons. First, because this bridge was built on the second Ring Road, the average vehicle flow is larger than on many other curved bridges. Second, the bearing forms of the bridge are spherical and basin rubber supported, and bearings have no limit on creeping, so creeping caused by centrifugal force should be restricted.

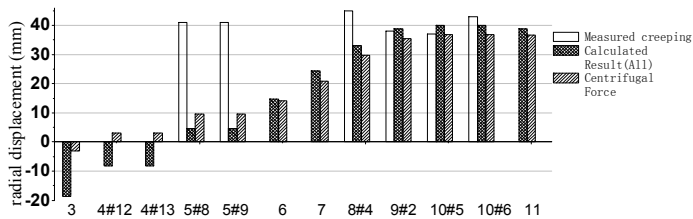


Figure 8. Creeping caused by centrifugal force.

Overall Analysis of Model Calculation Results. After calculation, the bridge radial displacements on the bearings are shown in Table 1.

The calculated results show that centrifugal force is the main culprit of creeping in TianNingSi (accounting for 80% of the creeping). Second is dead load and Prestress (accounting for 10% of the creeping). Temperature and creep and shrinkage effects accounting for another 10% of the creeping.

In summary, the deficiencies in infrastructure and bearing design should be the main cause of creeping disease. For wide bridges and large curve radius curved bridges in which the torsion effect is not obvious, bridge lateral stability is significant. Therefore, the double pier, pier-beam consolidation, and multi-bearing supporting pattern should be adopted in middle piers, as Figure 9(a) shows. The TianNingSi overpass has single piers in the middle pier, so the torsional capacity in the middle span is weak. This is detrimental to resisting creeping.

Table 1. Model Calculation Results on Radial Displacement.

No.	Dead Load	Pre-stress	Shrinkage Load	Creep Load	Temperature Fall	Centrifugal Force	Sum.	Measured Value
			20 Yrs	20 Yrs	25°C			
3axis	-2	-1	-3	-5	-4	-3	-19	
4axis No.12	-1	0	-2	-4	-3	3	-8	
4axis No.13	-1	0	-2	-4	-3	3	-8	
5axis No.8	0	2	-1	-3	-3	10	5	41
5axis No.9	0	2	-1	-3	-3	10	5	41
6axis	0	3	1	-2	-2	14	15	
7axis	1	3	2	-1	-1	21	24	
8axis No.4	1	2	1	0	-1	30	33	45
9axis No.2	1	2	1	0	0	35	39	38
10axis No.5	1	2	0	0	0	37	40	37
10axis No.6	1	2	0	0	0	37	40	43
11axis	1	2	-1	0	0	37	39	

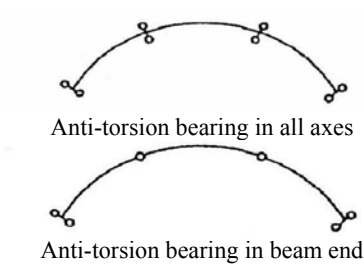


Figure 9. Supporting pattern in a curved bridge.

In addition, bearing form also has a great impact on creeping, including rational arrangement of different types of bearings and bearing eccentricity. Commonly-used methods for bearing arrangements are as follows. Torsional support can be set in both ends of the bridge, which can resist torque. Torsional support or hinge support can be used in mid-span support; as a result, torsion in the end of beam can be limited. In the TianNingSi overpass, bearing eccentricity is not set, and all the bearings in mid-span are not limited on creeping. This is detrimental to resisting creeping.

SOLUTION

This section discusses whether, when the TianNingSi overpass has restrictions of curvature radius, supporting form, and bearing eccentricity, creeping disease can be eliminated by changing bearing form.

The main function of bearing is to cause superstructure load such as dead weight, live load, etc., to transmit to the pier while ensuring that the superstructure can displace and rotate in a certain range. For the TianNingSi overpass, the bearings in the 4, 5, and 10 axes are basin rubber support, so bearings have no limit on creeping, resulting in some of the bearings emerging with the radial displacement transfinite disease.

To limit the creep volume, the 6, 8, and 12 bearing forms have been changed to unidirectional bearing GPZ10DX, which in JT 391-1999 means the bearing direction is unchanged.

Figure 10 shows that after changing the bearing form, the creeping volume is limited effectively, especially in the 6, 8, and 12 bearings. Therefore, setting unidirectional bearing is an effective way to limit creeping.

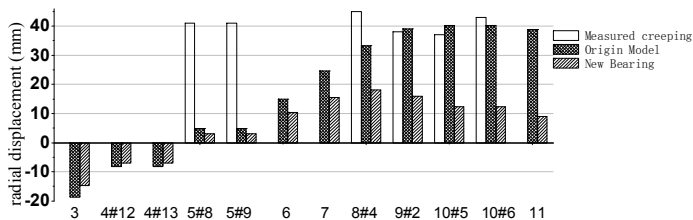


Figure 10. Trends of bridge radial displacement.

CONCLUSIONS

Through upper section analysis of the bridge model, we can obtain the creeping mechanism of a curved bridge as follows:

- 1) Creeping diseases are caused by geometric eccentricity, temperature load, prestressed effect, vehicle loads, and concrete shrinkage and creep effect. Centrifugal force is the main cause of creeping in a curved bridge that has heavy traffic and a large curvature radius.
- 2) The deficiencies in infrastructure and bearing form should be the main cause of creeping disease. The rational infrastructure of the double pier, pier-beam consolidation, and multi-bearing supporting pattern adopted in middle piers is the rational infrastructure for this bridge.
- 3) For this bridge, creeping diseases can be limited effectively by using unidirectional bearing in double pier.

ACKNOWLEDGMENTS

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Natural Vibration Properties Analysis of Continuous Rigid Frame Bridge Varying with Consolidation Damage at Pier Top

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ABSTRACT

In this paper, finite element models based on Wayaobao Bridge in Chongqing with a span combination of 71m+3×125m+71m are established using the finite element analysis software Midas/Civil 2010 to simulate the right-hand-side bridge. In the modeling, the action of self-weight, secondary dead load, prestressed load, and shrinkage and creep of the concrete, which may affect the natural vibration properties, is taken into account. With the concept of the consolidation damage degree being defined at the beginning, the natural vibration properties of the structure under different conditions with a series of damage degree of 0, 10%, 20%, 30%, and 100% at the side pier top were studied. What is shown from the comparative analysis of the results data is that the natural vibration properties of the whole structure are sensitive to the damage degree, and with the accumulation of the consolidation damage, stiffness of the structure continues to reduce. The process and the conclusion of the study may provide some reference for damage identification of bridge structures as well as similar studies on the connection damage.

Key Words: Continuous rigid frame bridge; consolidation damage; natural vibration properties; connection damage; damage identification

INTRODUCTION

Long-span and high-pier prestressed concrete continuous rigid frame bridges (LHPCFB) with many advantages are widely used, especially for high-grade highway crossings in mountainous regions. Geomorphology with big ups and downs where the bridge will be located makes LHPCFB the primary choice. With the number of high piers and long span bridges in steep ravine areas increasing, the main span and pier height increase correspondingly, and elevation differences between the piers grow, up to tens of meters maximum. However, because of pier height and elevation differences expanding, many seismic await, for example, higher requirements for the high pier's stability and the adjacent low pier's ductility in seismic action. In addition, study of how the

dynamic performance of the structure changes when the damage occurs at the connection between the main girder and the pier is still minimal. When a strong earthquake approaches, the type of bridge will suffer and then generally may be damaged in a form of bending deformation. At the same time, maximum curvature may occur in a wide range, so where the plastic hinge appears is uncertain. According to the relevant literature, in terms of the continuous rigid frame bridges, the side or shorter pier is more sensitive to vibrations than other piers and a plastic hinge is more likely to emerge. In that way, under the condition of different seismic peak accelerations, the pier top and pier bottom will be two main positions where the plastic hinge may occur.

CONSOLIDATION DAMAGE DEGREE (CDD)

Generally, in software simulation, consolidation connections in concrete structures are processed in accordance with infinite stiffness. That is, there is no relative angular displacement among the components connected to the node. But in reality, the value of connection stiffness of the node, even the rigid node, is finite. To avoid discussion of node connection stiffness, the bending stiffness of a member near the connection node was selected to replace it so as to simplify the problem. At the same time, pier damage occurs before the main girders, but only in the longitudinal direction, and no material aging effects resulting from time are included in the assumption.

In some previous studies, the concept of damage is vague, and its qualitative description cannot express the capability. It is necessary to propose a variable that can effectively characterize damage from a quantitative point of view. Zhang (2009) introduced a variable of material damage degree in a study on isotropic material to reflect the elastic modulus changes at different times. In the study, the concept of consolidation damage degree was introduced to characterize the damage at the connection of the main girder and the pier. The damage degree, D , can be expressed as follows:

$$D = \left(1 - \frac{M}{M_u}\right) \times 100\% \quad (1)$$

Where M_u is the bending capacity of the section in close to the pier top under initial conditions, and M is the bending capacity after the damage. According to material mechanics, when members reach their design capacity, there is:

$$\sigma_{\max} = \frac{M}{W} = [\sigma] \quad (2)$$

Where W is the bending modulus of the cross-section, and as for a rectangular cross-section, there is $W = (b \times h^2) / 6$; b is the width of the cross section, and h is the section height. Therefore, when the material is certain and the transverse cross-section changes are not taken into consideration, the section bending capacity (M) will change proportionally with the square of the effective bending height of the cross-section, that is, h^2 . Then the consolidation damage degree can

be expressed as:

$$D = \left(1 - \frac{h^2}{h_0^2}\right) \times 100\%$$

(3)

Where h_0 is the effective bending height of the cross-section in initial conditions and h is the effective bending height after damage.

ESTABLISHMENT OF FINITE ELEMENT MODEL

Wayabao Bridge is a curved bridge with a long span and high piers located on Yuxiang highway of China, whose main bridge structure form is a prestressed concrete continuous rigid frame system, with a span combination of 71m +3 × 125m +71 m. The main body is divided into two sides because of the separated foundation. The form of both sides' bridges is a composite curve of an R-2600-m circular curve combining with an A-1300 transition curve connecting with an R-4500-m circular curve. Taking only the right-hand-side bridge for example, the cross-section of the main bridge is a prestressed concrete single cell box section. Detail data of the cross-section of the main body are shown in Table 1. The main bridge has piers with the form of double thin-slender pier and bored pile foundation, whose specific section and layout are shown in Figures 1, 2, and 3.

Table 1. Detail Sizes of Beam Cross-Section.

Width	Beam top 12.25m	Beam bottom 6.5m	Two wings 2.875m
Height	Root of beam 7.5m	Middle 2.8m	End 2.8m
Thickness	Root of wing 70cm, end 15cm	Root of top plate 100cm, middle 30cm	Bottom plate 28cm, root of web plate 70cm, middle 40cm

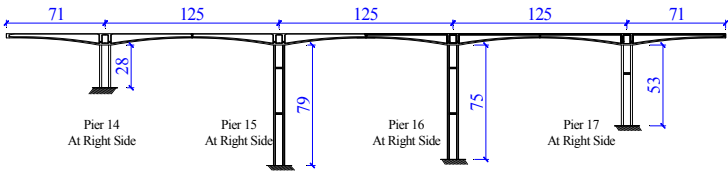


Figure 1. Overall facade arrangement drawing of right side bridge (m).

For numerical analysis, the finite element simulation model of the full right-side bridge has been established in using MIDAS/CIVIL 2010. In the model, the bridge is discretized into 341 nodes and 333 units. For convenience, the piers are numbered 14, 15, 16, and 17 from left to right, as shown in Figure 4.

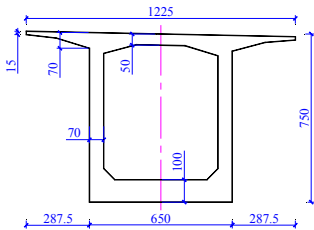


Figure 2. Cross-section of girder on pier top (cm).

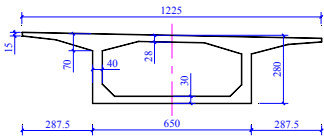


Figure 3. Cross-section of girder at midspan (cm).

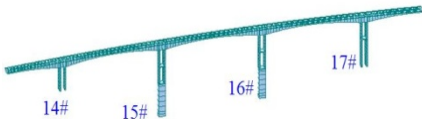


Figure 4. Full bridge simulation model.

Material: Concrete of the main girder is C55 in accordance with Chinese Specification, elastic modulus $E = 3.55 \times 10^4 \text{ MPa}$, density $Q = 2\,600 \text{ kg/m}^3$; that of pier C40, elastic modulus $E = 3.25 \times 10^4 \text{ MPa}$, density $Q = 2\,600 \text{ kg/m}^3$. Grade 270 of the high-strength low relaxation strands is used as prestressed reinforced.

Loads: The structure self-weight, second dead load, prestressing load, and material creep and shrinkage effects were taken into consideration in the model. The former three factors were also transformed into the structural mass, which may affect the vibration characteristics of the structure.

Analysis condition: Damage that occurs at the top of the left-side pier is treated as research contents, and five damage degrees of 0, 10%, 20%, 30% and 100% were studied in modeling.

During the processing, in order to simulate the rigid connection with finite value between the main bridge and pier, a connection pattern shown in Figure 5 was adopted. The left-side wire springs were used to replace the stiffness in the axial direction between nodes, the value of EA / L , and the right round springs to the bending stiffness of the original structure, whose value needed to be further calculated. In numerical analysis, changes of the consolidation damage degree can be easily satisfied by adjusting the rotational stiffness.

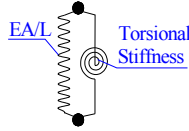


Figure 5. Schematic diagram of node semi-rigid connections.

DYNAMIC ANALYSIS

Eigenvalue computation theory. The finite element linear dynamic control equation of the structure with multi-degree of freedom is:

$$[M] \{\ddot{y}\} + [C] \{\dot{y}\} + [K] \{y\} = 0 \quad (4)$$

Ignoring the damping effect in a multi-degree of freedom system, the dynamic differential equation can be simplified as the form:

$$[M] \{\ddot{y}\} + [K] \{y\} = 0 \quad (5)$$

Assuming that free vibration of the system is simple harmonic motion, the formula can be expressed as $\{y\} = \{Y\} \sin(\omega t + \theta)$ and then the original formula can be converted to the form $\{[K] - \omega^2[M]\} \{Y\} = 0$. Then the natural vibration frequency of the structure can be solved one by one.

Where

$[M]$ = System mass matrix

$[C]$ = System damping matrix

$[K]$ = System stiffness matrix

$\{\ddot{y}\}$ = Accelerations array of system

$\{\dot{y}\}$ = Velocity array of system

$\{y\}$ = Displacement array of system

ω = Natural frequency of vibration modes

θ = Initial phase angle of each band of harmonic vibration

$\{Y\}$ = Amplitude of each vibration mode

Damage detection theory of bridge. Natural frequencies and modes of vibration characteristics of bridge structure similar to a human pulse can also be diagnosed. If the vibration characteristics of the structure have been obtained, the health status of the bridge can be determined. This prompted the birth of damage detection theory of bridges. Currently, structural fingerprinting theory is one of the most common theories in the field of the bridge damage identification. Structural fingerprinting, including static and dynamic fingerprinting, is the data information after processing displacement, stress, and other data that come from the dynamic and static load tests. Taking advantage of the mapping relationship existing between structural fingerprinting and the inherent characteristics of the structure, the practical situation of bridge structure can be certain after site testing.

Structural dynamic fingerprinting can be divided into four main categories: transferring characteristics; complex functions; passing curvatures; and characteristic parameters. Among them, frequencies and mode shapes are the two main driving force fingerprinting indexes in transferring characteristics. According to a free vibration characteristic equation without damping, it can be seen that frequencies and mode shapes are two factors that can directly influence the structural stiffness (generally assuming mass of the structure unchanged). Frequencies and mode shapes not only can be used as the quantitative indicators of changes in structural stiffness, but also to build a matrix inversion equation for inversion solution. For the natural frequency, it can be obtained directly through a dynamic test, whose test accuracy is relatively higher and easier, regardless of the position of the measuring point, and reflect the performance of the whole structure, all of which makes frequency the first choice of damage diagnosis. In contrast, vibration modes, especially the higher modes, are sensitive to changes in partial stiffness, so they can not only be used to locate the partial damage, but also to judge its degree. Obviously, it is the natural vibration frequencies and mode shapes that are the two most critical factors that need to be measured to judge and identify damage on a bridge structure.

RESULTS AND ANALYSIS

In the calculation process, the method of subspace iteration was adopted to analyze the dynamic characteristics of the bridge structure, and following are the five vibration modes with no consolidation damage at the pier top, as shown in Table 2.

Shown in Figure 6 and Figure 7 are, respectively, vibration frequency and cycle curves of the structural first 50-order under conditions with different consolidation damage degrees (CDD). What is shown is that frequency of the higher modes, generally greater than 30, is more sensitive to consolidation damage degrees, while the damage degree has a significant effect on the lower vibration period of vibration indicated in the figure of the cycle.

Table 2. First Five Vibration Modes of the Structure with Zero CDD.

				
Mode 1	Mode 2	Mode 3	Mode 4	Mode 5
First-order	First-order	First-order	Second-order	Third-order
lateral	vertical	longitudinal	lateral	lateral
vibration	vibration	vibration	vibration	vibration

To more clearly compare the different degrees of damage condition, in this paper taking the vibration cycle with the damage degree of 0% as the reference, the cycle difference curve was plotted in Figure 8, and the vibration frequency with the damage degree of 100%, the frequency difference curve is

shown in Figure 9. It is obvious that the natural cycle of the structure had the biggest change in value of 0.53s at mode 6 compared to before and after the damage. The biggest difference of frequency was 0.85Hz at mode 39. In general, the frequency had a great wave among the higher modes.

Subsequently, for the purpose of investigating the structure's natural frequency and cycle with changes in the degree of damage, mode 6 and mode 39 were selected, and then curves of the mode-6 cycle and the mode-39 frequency varying with the damage degree were plotted into Figure 10 and Figure 11. From the mode-6 figure, about 81% of the cycle change value is in the process of the damage degree from 0–10%, and the rest from 10–100%. Similar to mode 6, in the mode-39 figure, probably 75% of the frequency change value is in the process of the damage degree from 0–10%, then the rest from 10–100%. All the above shows that the structure vibration characteristics, including the cycle and the frequency, do not change with the consolidation damage degree in proportion. Most of all the changes are concentrated in a narrow range of less than 10% in consolidation damage degree.

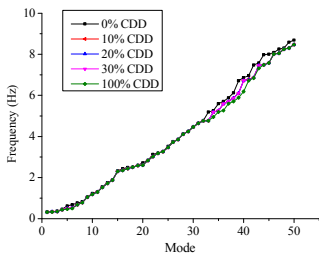


Figure 6. Mode-frequency curve of former 50 modes.

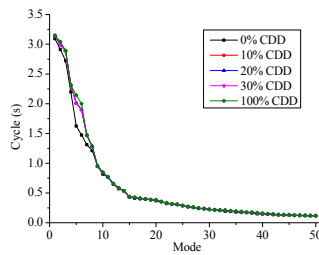


Figure 7. Mode-cycle curve of former 50 modes.

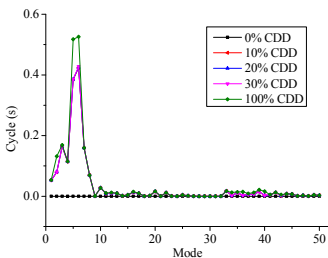


Figure 8. Mode-cycle-difference curve of former 50 modes.

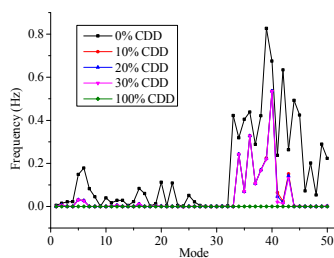


Figure 9. Mode-frequency-difference curve of former 50 modes.

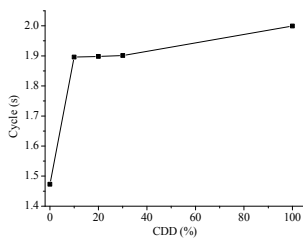


Figure 10. Mode 6 vibration cycle curve.

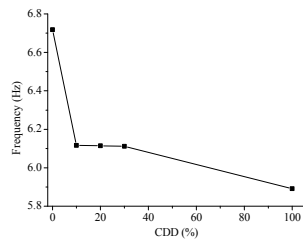


Figure 11. Mode 39 vibration frequency curve.

CONCLUSION

In this paper, through simulation and analysis of Wayaobao SuperBridge in China using Midas, some conclusions can be safely drawn as follows:

- 1) Because of the of consolidation damage degree in concrete structures, damage at semi-rigid connections gets a quantitative description in degree.
- 2) From the representative vibration mode dynamic curves, a mutation occurs in vibration characteristics of the structure, when the damage degree of the concrete connection goes from initial zero to a very small value.
- 3) Changes in the value of the structure’s natural vibration frequencies do not vary with the consolidation damage degree in proportion. When the initial rate is large, there is sharp attenuation. In the way, when the consolidation damage degree of a structure connection is large, it is not desirable to conduct structure damage identification and diagnostics by frequency change, since the natural frequency is no longer sensitive to the degree.

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Dynamic Responses of Rail-Embankment-Foundation on High-Speed Railways under Moving Loads

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ABSTRACT

Movement of high-speed trains is modeled by moving load technology; radiation damping and elastic recovery of infinite foundations are simulated by a three-dimensional viscoelastic static-dynamic unified artificial boundary. By considering the influence of the static stress state after embankment and rail system construction on subsequent dynamic analysis, material nonlinearity and dynamic interaction between reinforced concrete roadbeds and the upper layer of formation, large-scale three-dimensional dynamic parallel computation of a rail embankment foundation system on a high-speed railway was performed. Results including vibration displacement, vibration velocity, vibration acceleration, and dynamic stresses were obtained. On these bases, an attenuation law of vertical dynamic stress with depth was analyzed and the influence depth of dynamic load was determined. It was found that the principal stress axes in the three planes rotated continuously during the period of load moving, and the rotation in YZ plane was dominant. The rotation of the principal stress axes was compared with that of a soil element induced by the moving load applied directly to the foundation surface.

INTRODUCTION

In the past decade, high speed rail has developed quickly in China. High speed trains make for fast travel and are comfortable and convenient. On the other hand, they pose problems such as high-level vibrations. With the increase of running speed, the dynamic effects generated by high speed trains become much more prominent

compared to middle or low speed trains, thus bringing challenges for the design and maintenance of high speed railways. It is necessary to conduct detailed studies on the dynamic behaviors of substructures under high-speed moving loads.

In this study, a full-size true three-dimensional precisely nonlinear model was established to investigate the dynamic effects of a rail-embankment-foundation system under impulse loads with moving speeds of 350km/h.

FINITE ELEMENT MODEL

Model description. The finite element model is shown in Figure 1; the length, width, and height are 450m, 120m, and 107.8m, respectively. The model consists of rail, sleeper, and fastener, cement asphalt (CA) mortar, reinforced concrete (RC) roadbed, upper layer of the formation, lower layer of the formation, embankment below the formation, and foundation. The distance between the two sleepers in the longitudinal direction is 0.625m. Strict requirement on mesh size is needed in dynamic analysis to perform stable analysis, so the mesh size of the rail should be small, but this may drastically increase the calculation scale. For this reason, only the rail, sleeper, fastener, track slab, CA mortar, and RC roadbed on one way were included in this model.

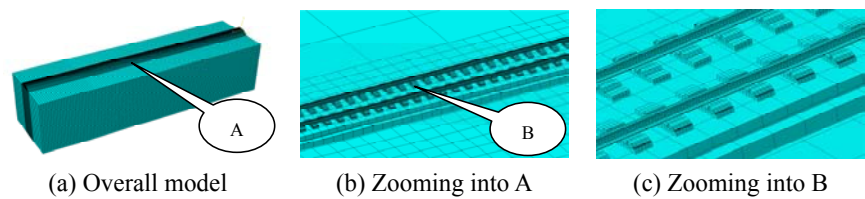


Figure 1. Analysis model.

Model parameters. According to a geotechnical survey and laboratory test, the parameters for foundation’s soil mass are listed in Table 1. Rayleigh damping coefficients were obtained based on the damping ratio and natural frequencies of the whole model. A Drucker-Prager constitutive model was adopted for soil mass.

The interface between the reinforced concrete roadbed and the upper layer of the formation was very coarse, so contact pairs were adopted to model their dynamic interaction. Coefficient of friction on the contact surface can be determined by $\mu=\tan\delta=2/3\tan\phi$ (Zhang 2010).

Parameters for the embankment and rail system are listed in Table 2 and Table 3, and a linear elastic constitutive model was adopted.

Table 1. Parameters for Foundation Soil Mass.

Name	Unit wt.	Compr. modulus	Poisson's ratio	Cohesion	Int. friction angle	Damp. ratio	Coefficient of Rayleigh damping	
							α	β
	$\text{kN}\cdot\text{m}^{-3}$	MPa	-	kPa	°	-	s^{-1}	s
Clay	18.0	9.2	0.30	22.0	16.0	0.08	0.262	0.024
Clay	18.6	11.3	0.28	25.2	18.1	0.07	0.229	0.0214
Silty clay	19.3	13.2	0.29	24.6	17.6	0.065	0.213	0.0199
Silt	19.8	16.5	0.25	26.1	20.5	0.055	0.180	0.0168

Table 2. Parameters for Embankment.

Name	Unit wt.	Compr. modulus	Poisson's ratio	Cohesion	Int. friction angle	Damp. ratio	Coefficient of damping ratio	
							α	β
	$\text{kN}\cdot\text{m}^{-3}$	MPa	-	kPa	°	-	s^{-1}	s
Upper layer of formation	19.50	190.0	0.3	32.0	75.0	0.08	0.262	0.024
Lower layer of formation	19.00	100.0	.0.3	26.0	25.0	0.07	0.229	0.021
Embankment below formation	18.50	85.0	0.28	25.0	22.3	0.10	0.328	0.031

Table 3. Parameters for Rail System.

Name	Unit wt.	Elastic modulus	Poisson's ratio	Damping ratio	Coefficient of Rayleigh damping	
					α	β
	$\text{kN}\cdot\text{m}^{-3}$	GPa	ν	ξ	s^{-1}	s
Rail	78.0	206.0	0.30	0.01	0.0328	0.0031
Track slab	24.0	35.5	0.10	0.03	0.0983	0.0092
Sleeper	24.0	30.0	0.10	0.03	0.0983	0.0092
Concrete base	24.0	30.0	0.10	0.03	0.0983	0.0092
CA mortar	18.0	0.10	0.40	0.05	0.0983	0.0092
Fastener system	Dynamic stiffness: 60kN/mm, coefficient of damping: 8kN·s/m					

Boundary condition. A three-dimensional viscoelastic static-dynamic unified artificial boundary (Liu 2005) was adopted in the analysis. This special artificial boundary has a high effectiveness and precision simulating radiation damping and

elastic recovery of the infinite foundation. The influence of the static stress state on the dynamic analysis was taken into account by this artificial boundary.

Load condition. In the analysis, the interaction force between the wheel set and the rail was taken as pressure and modeled by impulse load. The impulse load was applied directly to the loading area on top of the rail and can be expressed as:

$$p = P \sin^2 (2\pi ft)$$

Where p = pressure in contact area, P = maximum pressure in contact area, and f = load frequency.

Based on the EMU train dimensions, running speed, and loading area, the impulse load can be written as:

$$p = 96.36 \times \sin^2 (2\pi \times 5 \times t) \quad \text{MPa}$$

The plot of the impulse load is shown in Figure 2.

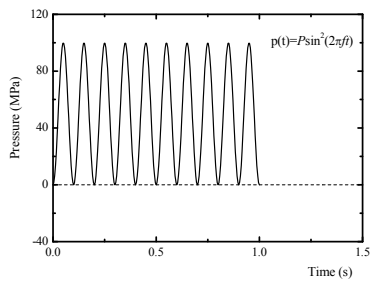


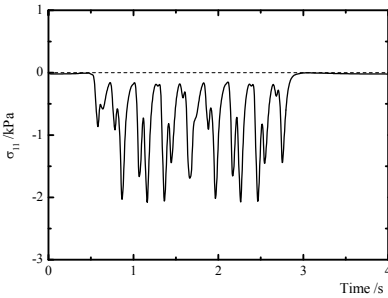
Figure 2. Impulse load.

Total calculation time for dynamic analysis was set to 4.0 seconds.

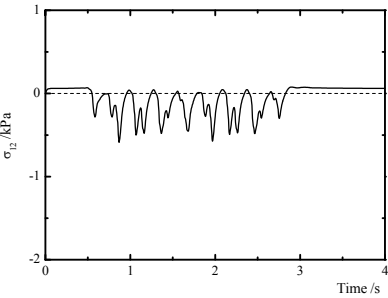
RESULTS ANALYSIS AND DISCUSSIONS

Dynamic stresses in upper layer of formation. Elements were selected before calculation. Time histories of the dynamic stresses for the selected element under one of the rails in the upper layer of the formation are shown in Figure 3.

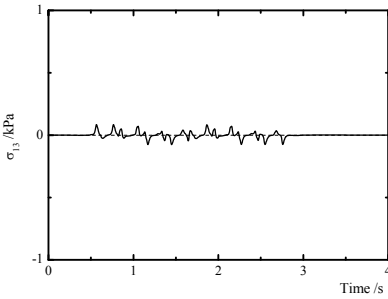
It can be seen from Figure 3 that under vertical moving load, six stress components of the selected element were not equal to zero and vary rapidly with time. The vertical dynamic stress component was predominant and had a maximum value of 10.8kPa. The normal stress components were symmetric, and the stress components σ_{13} and σ_{23} were anti-symmetric.



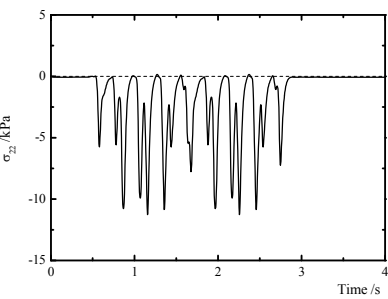
(a) σ_{11} time history



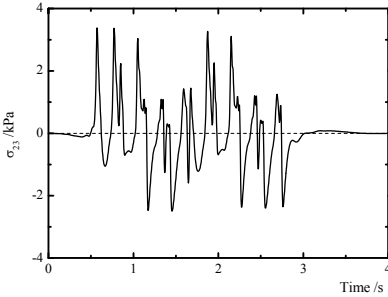
(b) σ_{12} time history



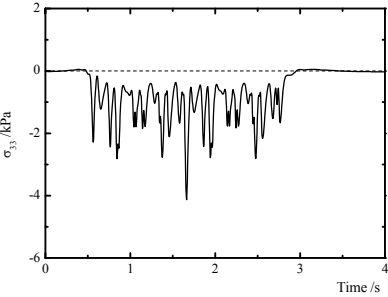
(c) σ_{13} time history



(d) σ_{22} time history



(e) σ_{23} time history



(f) σ_{33} time history

Figure 3. Dynamic stresses in upper layer of formation.

Based on the above stress components, it was concluded that the selected element was in a state of compression. It experienced loading and unloading repeatedly during the load moving. This stress state could easily cause material fatigue and even permanent deformation, so special attentions should be paid in embankment design.

Distribution of vertical velocity in rail-embankment-foundation system. From the analysis results, it was found that the vertical response as predominant, so only the vertical velocity was analyzed in this paper due to page limitations. The vertical velocity distribution at 1.160s was taken as an example, as shown in Figure 4.

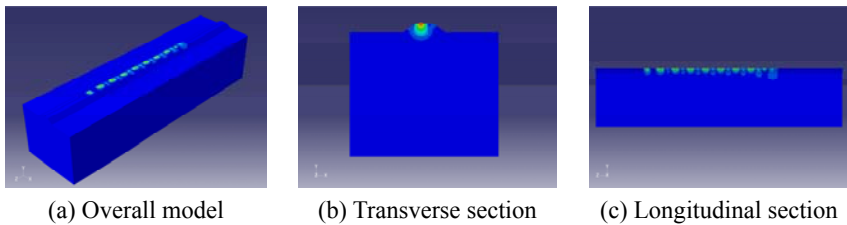


Figure 4. Distribution of vertical velocity at 1.160s.

It can be seen from Figure 4 that the vertical velocity at the computational boundaries is zero because all the waves reaching the computational boundaries are absorbed by the special artificial boundary mentioned before; no wave was reflected into the calculation domain, which ensures the correctness and validity of the results. The distribution of the vertical velocity was limited to a relatively small region on the model. The locations with maximum values correspond to the load in space, and the influence range of the dynamic load is relatively small. The speed of the moving load in this analysis was 97.2 m/s, which was less than the shear wave speed of the upper layer of the formation. It can be seen clearly that the wave front was in front of the location where the load was applied (Figure 4 (c)).

Distributions of vertical dynamic stresses along depth. The distributions of vertical dynamic stress along depth at different times of 0.888s, 1.160s, 1.636s and 1.960s are shown in Figure 5.

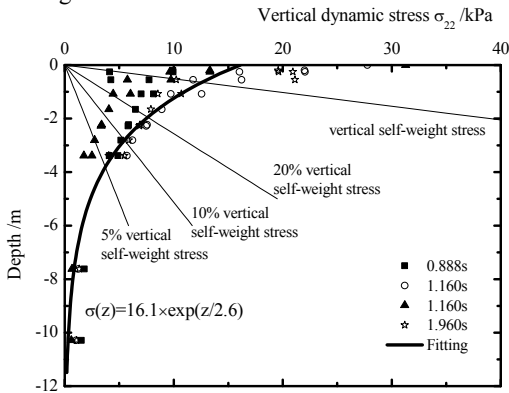


Figure 5. Distribution of vertical dynamic stresses along depth.

Distributions of vertical dynamic stress at four different times were analogous except for different maximum values varying between 12.9kPa and 31.3kPa. Attenuation law of vertical dynamic stress with depth can be predicted by exponential function as:

$$\sigma(z) = 16.1 \times \exp\left(\frac{z}{2.6}\right)$$

where, $\sigma(z)$ = vertical dynamic stress at depth of z , and z = depth.

To determine influence depth of the dynamic load, the vertical dynamic stress was compared with the 20%, 10% and 5% vertical self-weight stresses (see Figure 5). If the depth where the vertical dynamic stress equals 20% vertical self-weight is taken as the influence depth of the dynamic load, then the depth was approximate 0.7m below the upper layer of the formation. The influence depth corresponding to the 10% and 5% vertical self-weight stresses are 1.9m and 2.9m below the upper layer of the formation, respectively.

Rotation of principal stress axes. The stress paths of an element in the upper layer of the formation were analyzed, as shown in Figures 6, 7, and to 8. For comparison, the stress path of a soil element at 0.7m below foundation surface is presented. In the foundation model, one load with a moving speed of 97.2m/s was applied directly to its surface.

As can be seen from Figures 6, 7, and 8, the three axes of the principal stresses rotated continuously during load(s) pass through, and the rotation in the YZ plane was predominant. In this analysis, the speed of the moving load was less than the minimum shear wave speed of rail-embankment-foundation system, so the stress state of the selected element was affected firstly by wave motion, then by the moving load, so the rotation did not start from origin of coordinate. Due to the existence of the rail system, the rotation of the principal stress axes was much more complicated than that of foundation soil element. Rotation of the principal stress axes was one of the most important factors causing permanent deformation; the rotation effects must be considered in the design of embankments for railway and highway.

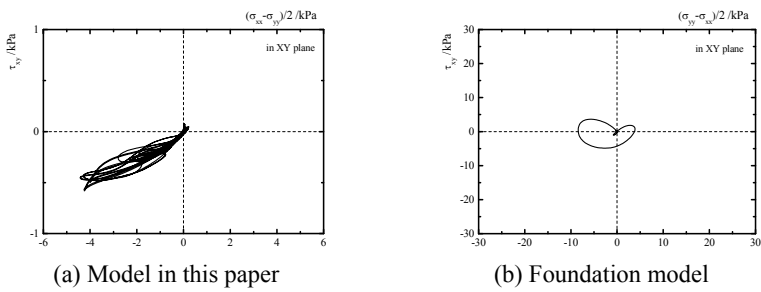


Figure 6. Rotation of principal stress axes in XY plane.

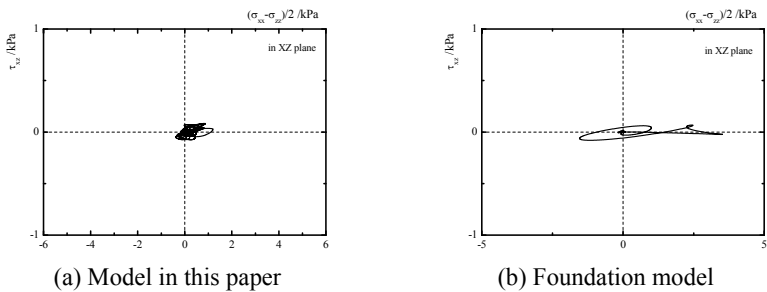


Figure 7. Rotation of principal stress axes in XZ plane.

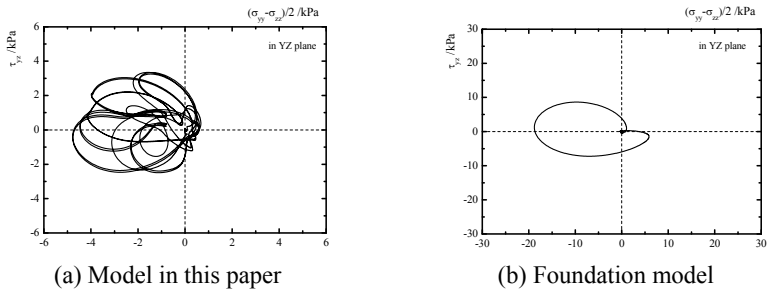


Figure 8. Rotation of principal stress axes in YZ plane.

CONCLUSIONS

Based on the FE analysis results, the following conclusions are drawn:

- 1) The rail-embankment-foundation system is mainly in a state of compression.
- 2) The influence range of the dynamic moving loads is limited.
- 3) The stress state of the soil element is affected first by wave motion, and then the principal stress axes rotate continuously during load pass-through.

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Study on the Distribution of Coarse Aggregate in Concrete

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ABSTRACT

The distribution of coarse aggregate in concrete at different depth from the surface layer was investigated. Based on the fractal theory, the fractal dimension model of the coarse aggregate in concrete was induced. Meantime, the cross sectional area percentage of the coarse aggregate in concrete was calculated by sectioning method, and the fractal dimension value was also indirectly estimated. The results indicate that there was certain change regularity of the coarse aggregate content in concrete, which presents a low percentage value in the surface region and a high percentage value at the depth. That may be caused by the effect of the size effect and boundary effect during the process of the production. In addition, the fractal dimension value of the coarse aggregate in concrete is about 1.35, which differs from the traditional Euclidean geometry. That may be due largely to the coarse aggregate in concrete with fractal character.

KEYWORDS: concrete; coarse aggregate; fractal; distribution

INTRODUCTION

As the important composition of the concrete, the characteristics of the coarse aggregate can obviously impact the macro properties of the concrete. Much of research has been carried out to discuss the influence of the coarse aggregate on the properties of the concrete, and lots of plentiful fruits have been acquired (Mora et al 2000). For example, Huang et al (Huang et al 2012) studied the influence of the coarse aggregate on the carbonization resistance of the concrete, and the results revealed that the depth of the carbonation in concrete cover decreased with increasing of the coarse aggregate dosage. Yang et al (Yang 2003) investigated the effect of the dosage of coarse aggregate on the chloride ion migration coefficient, and reference (Shen 2010) analyzed the relations of the the coarse aggregate forms on the interfacial transition zone of the mortar and coarse aggregate. Moreover,

the kinds of the coarse aggregate on the mechanical properties of self-compacting concrete were also illustrated by Mucteba (Mucteba 2012). However, the above results were mostly focused on the size and kinds of the coarse aggregate, and the distribution of the coarse aggregate in concrete was rarely discussed. Generally speaking, the distribution of the coarse aggregate regarded as the important composite is agreement with the fractal geometric characteristics. In fact, the coarse aggregate possesses expect performance due to its high density, which endows it hard to corrode. Therefore, the mortar and the interfacial transition zone are the main transmission paths for the harmful substance, such as chloride ion and sulphate. Because the distribution of the coarse aggregate in concrete can change the characteristics of the mortar, so the study of coarse aggregate distribution in concrete is great significance. At present, fractal regarded as the novel method is gradually used in the civil engineering field. Usually, the fractal is used to describe the self-similar graphics and the structure which have certain significance but without characteristic length (Mandelbrot 1982). Concept of the fractal was proposed by Mandlbrot in 1975, and the quantitative parameters for characterization of fractal is named fractal dimension (Ostoja 1989). As we know, the macro properties of the concrete are uncertainty, irregular and nonlinear, which embodies the complicated properties of the micro structure. Much of the research on the concrete crack and pore fractal was carried out at home and abroad (Panagiotopoulos 1992); For example, the Thompson and Katz (Thompson 1987) investigated the pore structure of the porous materials was with fractal characteristics by scanning electronic microscope (SEM). Tang et al (Tang 2002) studied the fractal characteristics and the topological property of the concrete. However, the fractal of the coarse aggregate distribution in concrete has rarely involved.

In this article, the distribution of coarse aggregate in concrete at different depth from the surface layer was investigated. Based on the fractal theory, the fractal dimension model of the coarse aggregate in concrete was induced. Meantime, the cross sectional area percentage of the coarse aggregate in concrete was calculated by sectioning method, and the fractal dimension value was also indirectly estimated, which was a novel method to discuss the durability of the concrete.

THEORETICAL MODEL

Based on the fractal theory, the complexity and irregular of the set is described by fractal dimension regarded as the main index of the image characteristics. Different objects in nature generally have different fractal dimension corresponding to the image, which represents different physical property. Many kinds of methods used to obtain the fractal dimension are box-counting method, perimeter-size method and transform method. Correspondingly, there are many kinds of fractal dimension including hausdorff dimension, box dimension, Lyapunov dimension, and so on (Wu et al 2007; Xie et al 1997). In fact, the

box-counting method is widely used to calculate the fractal dimension, which is due to its simple and accurate property.

Fractal geometric Equation was usually used to describe the fractal characteristics of the objects, listed as Equation (1).

$$A(\delta) = A_0 \left(\frac{\delta}{\delta_{max}} \right)^{E-D} \quad (1)$$

Where A and δ respectively stand for points and segments when E is zero. A and δ respectively represent length when E is one. A and δ respectively delegate area and length when E is second. If the E was three, the A and δ respectively stand for volume and length. A_0 is the value of A , when the figure is plastic dimension. δ_{max} is the maximum length of yardstick, and D is the fractal dimension.

The distribution of the coarse aggregate in concrete was regarded as a certain function, listed as Equation (2).

$$F(x) = \frac{N(x)}{N_0} \quad (2)$$

Where $F(x)$ is the function of the distribution of coarse aggregate; $N(x)$ stands for the total number of coarse aggregate with the size less than x . N_0 is the total number of the coarse aggregate in concrete, and x is the size of the sifter.

Based on the Equation (1) and (2), the fractal of the graded aggregate size distribution can be obtained as Equation (3).

$$F(x) = \left(\frac{x}{x_{max}} \right)^{-D} \quad (3)$$

If the mass function of the aggregate distribution can be listed as Equation (4), and the volume of the coarse aggregate can be described as Equation (5), then the fractal of the graded aggregate mass distribution can be obtained under the boundary conditions ($P(x_{max})=1$ and $P(x_{min})=0$), listed as Equation (6).

$$P(x) = \frac{M(x)}{M_0} \quad (4)$$

$$V = k_v x^3 \quad (5)$$

$$P(x) = \frac{x^{3-D} - x_{min}^{3-D}}{x_{max}^{3-D} - x_{min}^{3-D}} \quad (6)$$

Where $M(x)$ is the mass of the coarse aggregate with size no more than x , M_0 is the total mass of the coarse aggregate in concrete, and the $P(x)$ stands for the passing rate of the coarse aggregate with size less than x . The k_v is the shape factor of the smooth aggregate volume.

The dimension of the single coarse aggregate was three rather than fractal, but the coarse aggregate mixed together cannot fill the entire volume, which leded the appearance of the void. So the insufficient space filling ability caused

the presence of the volume fractal. Based on the Equation (1), the volume fractal of the graded aggregate can be listed as Equation (7).

$$V = V_0 \left(\frac{x}{x_{max}} \right)^{3-D_v} \tag{7}$$

Where V is the fractal volume of the coarse aggregate, V_0 is the plastic dimension, and D_v is fractal of the volume.

If the volume can be defined as Equation (8), then the total fractal volume V which was integral from x_{max} to x_{min} can be listed as Equation (9). Meanwhile, the Equation (6) and (7) were also be used to obtain the corresponding void listed as Equation (10). Based on the Equation (10) and (11), the fractal dimension of the coarse aggregate distribution in concrete can be obtained by fitting curve.

$$dV_0 = \frac{M_0}{\rho} dP \tag{8}$$

$$V = \frac{3-D}{6-D-D_v} \frac{M_0}{\rho} \left[\frac{x_{max}^{6-D-D_v} - x_{min}^{6-D-D_v}}{x_{max}^{3-D} - x_{min}^{3-D}} \right] x_{max}^{D_v-3} \tag{9}$$

$$V_{oid} = \frac{\frac{M_0}{\rho} - V}{\frac{M_0}{\rho}} = 1 - \frac{3-D}{6-D-D_v} \left[\frac{x_{max}^{6-D-D_v} - x_{min}^{6-D-D_v}}{x_{max}^{3-D} - x_{min}^{3-D}} \right] x_{max}^{D_v-3} \tag{10}$$

$$V_{oid} = 1 - \frac{\rho}{\rho'} \tag{11}$$

Where M_0 is the total mass of coarse aggregate, P is the passing rate of the coarse aggregate, the ρ is the bulk density of the coarse aggregate, ρ' is the real density of the coarse aggregate. In addition, the V_{oid} is the void of the graded coarse aggregate, which can also be calculated by Equation (11).

EXPERIMENT PROCESS

Grade 42.5 Portland cement, polycarboxylic type high performance water reducer, class I fly ash, grade S95 slag, water, sand and limestone with the size of 5 mm to 20 mm were used as the main raw materials to produce grade C40 concrete. The real density and bulk density of the limestone respectively is 2.63g/cm³ and 1.76 g/cm³. Mass ratio of the limestone, sand, cement, water, slag, fly ash and water reducer is 1060:710:355:155:70:45:4.5, and the size of the concrete sample is 150 mm×150 mm×150 mm. Profile grinding machine with the model of PF1100 produced by Denmark was used to prepare the sample, and the grinding hole diameter was 80 mm. In addition, the box-counting method was employed to calculate the area of the coarse aggregate. The profile grinding machine was as shown in Figure 1.



Figure 1. Profile Grinding Machine.

RESULTS AND DISCUSSION

In order to discuss the change law of the coarse aggregate distribution with depth from the concrete surface, the area ratio of the coarse aggregate at different depth in concrete was tested by testing. The profile of the concrete sample was prepared, and the content of the coarse aggregate at different depth in concrete was as shown in Figure 2.



Figure 2. Distribution of the Coarse Aggregate in Concrete.

From Figure 2, it was seen that the distribution of the coarse aggregate in concrete showed remarkable law. There was little coarse aggregate in concrete surface zone, and much of coarse aggregate distributes in concrete inner zone. Although there was a little coarse aggregate at certain depth, but most of the harden system was mortar. With increase of the depth from the concrete surface, the content of the coarse aggregate increased and tended to a constant. That was due to the effect of the size effect and boundary effect during the process of the production. Figure 2 also showed that the coarse aggregates surrounded by mortar were homogeneous dispersion in concrete, and they didn't contact together.

In view of the results shown in Figure 2, the profile grinding machine employed to prepare the sample was used to discuss the distribution of coarse aggregate at different depth from the concrete surface. Figure 3 showed the content of the coarse aggregate at different depth in concrete.

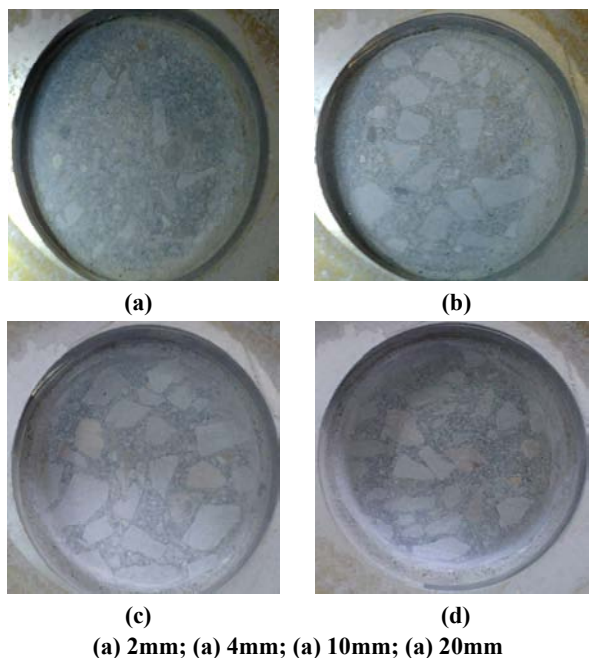


Figure 3. Content of Coarse Aggregate at Different Depth in Concrete.

Figure 3 revealed that the distribution of the coarse aggregate in concrete at different depth presented obvious law, which manifested the coarse aggregate content was little at 2 mm zone shown in Figure 3 (a). With increase of the depth, the coarse aggregate content increased quickly. When the depth was more than certain value, the profile area ratio tended to a constant as shown from Figure 3 (b) to Figure 3 (d), which accorded with the Figure 2. In order to investigate the quantitative characterization of coarse aggregate distribution in concrete, box-counting method was adopted to assess the coarse aggregate area ratio in concrete at different depth. Figure 4 showed the area ratio of the coarse aggregate at different depth.

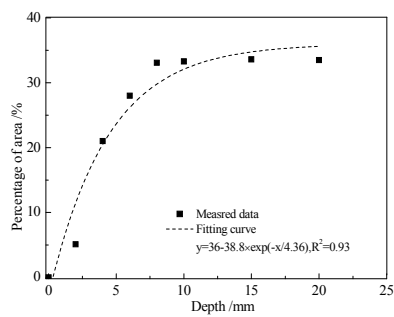


Figure 4. Area Ratio Curve of the Coarse Aggregate at Different Depth.

From Figure 4, it was seen that there was difference of the coarse aggregate area ratio at different depth in concrete, which represented the value increasing significantly with the depth from concrete surface. The area ratio curve embodied the change of coarse aggregate distribution, which can be manifested by exponential function. Figure 4 also revealed that the most significant change zone of the coarse aggregate in concrete was at 5 mm zone, which was due to the boundary effect between concrete surface and formwork. In order to investigate the characteristics of the coarse aggregate distribution, the inner zone within 2.5 mm from the concrete surface was regarded as homogeneous system of mortar, and the fractal dimension of the coarse aggregate total volume in concrete was calculated. Figure 5 was the curve of the graded coarse aggregate in concrete.

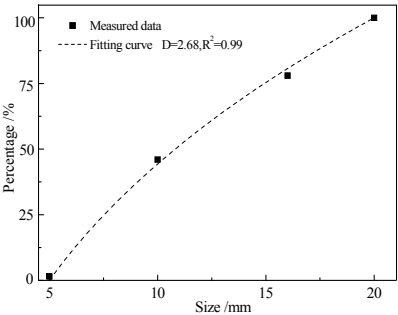


Figure 5. Curve of the Graded Coarse Aggregate in Concrete.

Figure 5 shown that the graded coarse aggregate used in concrete had well-graded, and the measured data was fitted to the curve. Based on the Equation (6), the fractal dimension of the graded coarse aggregate in concrete was about 2.68, and the correlation coefficient R was very high. It was also revealed that the fractal dimension model of the graded coarse aggregate was reasonable. Based on the results above, the side length of the sample used to calculate the fractal dimension was about 147.5mm, and the coarse aggregate was regarded as evenly distributed in concrete. The equivalent void of concrete was about 0.576, which was based on the assumption that mortar and the pore in concrete were regarded as void. So the corresponding fractal dimension was about 1.35, which was different from the viewpoint of Euclidean geometry regarding the volume of coarse aggregate in concrete was three dimensional. Generally speaking, the coarse aggregate in concrete cannot fill the whole space, which endows it fratal characteristics. Therefore, the coarse aggregate distribution in concrete has lower fractal dimension.

CONCLUSION

The coarse aggregate distribution in concrete at different depth was investigated by profile sectioning method. The results indicated that there was certain change regularity of the coarse aggregate content in concrete, which presented a low

percentage value in the surface region and a high percentage value at the depth. That may be caused by the effect of the size effect and boundary effect during the process of the production. In addition, the fractal dimension value of the coarse aggregate in concrete was about 1.35, which differed from the traditional euclidean geometry.

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Study on Application of Asaoka Method on Settlement Prediction of Pile Foundation for High Speed Railway Bridges

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ABSTRACT

Asaoka method is one of the main methods in settlement prediction of pile foundation. Asaoka method estimates final settlement data according to an unknown data calculated from the actual settlement observation. It is simple, and has high accuracy and reliability features in the prediction. For this reason, Asaoka method has been widely used because of the good prospects forecast in the pile settlement. However, there are also two shortcomings of Asaoka method: the choices of starting time influence the correlation coefficient and completion rates, time interval also has a direct impact on the final settlement prediction result. Based on the large field testing settlement data of pile foundation of high speed railway bridge, we can make the predictions get closer to the actual results through the suitable choice of starting time and time interval.

INTRODUCTION

High-speed railway pile settlement is one of the important factors that affect the safety of high-speed railway bridges. So the high-speed railway bridge foundation settlement observation is very important. Through the settlement observation data analysis, we can investigate current trends and predict the final settlement. Before obtain the reliable mechanical model, mathematical formulas can be used to fit the data. But it needs careful study on the form and applicability of the fitting formula, not simply judge the rationality according to the fitting accuracy. Choosing complex formulas and parameters to be determined in pursuit of excessive high fitting accuracy may make fitting formula parameter variation represented completely distorted.

High-speed railway pile settlement prediction model should not pursue too many parameters and complex formula to improve the prediction accuracy, but should be combined with a large amount of measured data to establish settlement prediction model which has a clear physical meaning. Asaoka formula would meet this requirement. Its parameters have clear physical meanings. Therefore, it has been widely used in engineering projects.

SHORTAGE OF ASAOKA METHOD

Asaoka method estimates final settlement data according to a unknown data calculated from the actual settlement observation. However, when use Asaoka method to predict the pile settlement, there are some deficiencies: Asaoka method using recursive method to establish settlement prediction expressions, recurrence is starting from the initial settlement observations, the choice of starting time cause the correlation coefficient and the percentage of completion method have large deviations, while the time interval has a direct impact on the final predictions

THE CHOICE OF STARTING TIME

In the projection process of Asaoka method, the value Δt has a directly impact on the final prediction results. Δt too small will cause greater volatility in the fitting point, the correlation coefficient of the fitted line will be small: If Δt is too large, Si point is less, then will cause a large deviation. Thus, in the actual projection process, we should be calculated more different Δt at the same time to draw the corresponding value of the final settlement, then select the better settlement value of the correlation coefficient as the final settlement value.

In this section, we use the Beijing-Shijiazhuang Passenger Line Yongding River Bridge [DK30 +600 ~ DK30 + 731] section B292[#] (measuring point 0030600D1) pier settlement observation data as a foothold for system analysis. Measuring point 0030600D1 time-settlement measured curve shown in Figure 1.

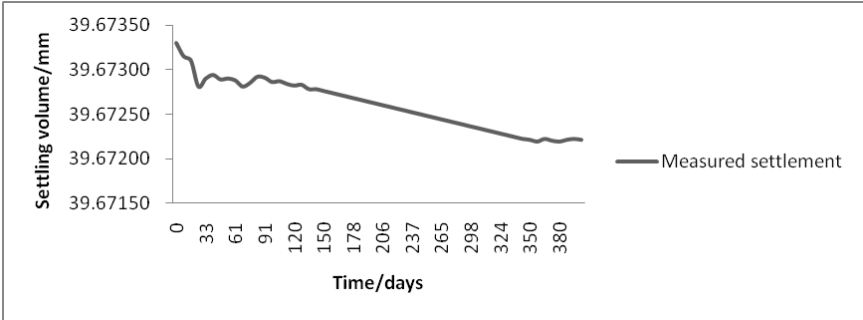


Figure 1. Measuring point 0030600D1 time-settlement measured curve

Measuring point 0030600D1, actual measurement starting time is December 19, 2009, using of Asaoka method to calculate final settlement, settlement after construction, correlation coefficient, completion rates when the starting point is December 19, 2009, December 26, 2009, January 2, 2010 and March 3,2010 , as shown in Table 1.

Table 1. Calculation results of 0030600D1 measuring points of different time origin

item	2009-12-19	2009-12-26	2010-01-02	2010-02-10	2010-03-13
final settlement	1.071	1.063	1.043	2.665	1.321

settlement after construction	0.058	0.166	0.192	1.986	0.610
correlation coefficient	0.915	0.943	0.943	0.989	0.995
completion rates	94.574	84.413	81.574	25.495	53.820

As shown on Table 1, select of different time will lead to different calculations, so that the results have certain discrete. At the same time interval, the value of the initial time t_0 more backward, correlation coefficient is higher, the settlement after construction is higher, the percentage of completion is lower, which will take longer to meet the requirements. Thus, workers after settling time to meet the next process are longer.

THE CHOICE OF TIME INTERVAL

This section use the Beijing-Shijiazhuang Passenger Line Yongding River Bridge [DK30 +600 ~ DK30 + 731] section B292 # (measuring point 0030600D1), B293 # (measuring point 0030633D1), B294 # (measuring point 0030665D1), B295 # (measuring point 0030698D1), B296 # (measuring point 0030731D1) a total of five piers settlement observation data as a foothold for system analysis.

First, using of Asaoka method to calculate final settlement, settlement after construction, correlation coefficient, completion rates when the time interval is 7 days, 14 days, 21 days, 28 days, 35 days condition, measuring points 0030600D1, 0030633D1, 0030665D1, 0030698D1, 0030731D1, as shown in Table 2. And the measuring point’s time-measured settlement curve shown in Figures 2 to 6.

Table 2. Calculation results of different time intervals

measuring point	item	7 days	14 days	21 days	28 days	35 days
0030600D1	final settlement	1.071	1.147	0.995	1.172	1.195
	settlement after construction	0.058	0.110	0.030	0.139	0.150
	correlation coefficient	0.915	0.949	0.884	0.952	0.972
	completion rates	94.574	90.437	97.009	88.104	87.448
0030633D1	final settlement	0.992	0.960	0.905	1.206	1.321
	settlement after construction	0.145	0.129	0.097	0.335	0.442
	correlation coefficient	0.919	0.924	0.907	0.936	0.973
	completion rates	85.333	86.541	89.286	72.221	66.508
0030665D1	final settlement	1.416	1.845	1.196	3.174	1.985
	settlement after construction	0.548	0.969	0.352	2.278	1.112
	correlation coefficient	0.948	0.967	0.925	0.967	0.970
	completion rates	61.324	47.454	70.583	28.240	43.956
0030698D1	final settlement	1.437	1.493	1.439	1.613	1.491
	settlement after construction	0.150	0.199	0.162	0.309	0.200
	correlation coefficient	0.990	0.994	0.989	0.995	0.995

	completion rates	89.532	86.638	88.721	80.843	86.565
0030731D1	final settlement	1.481	1.549	1.605	1.845	1.622
	settlement after construction	0.224	0.288	0.341	0.566	0.357
	correlation coefficient	0.991	0.995	0.994	0.998	0.998
	completion rates	84.864	81.402	78.735	69.320	77.974

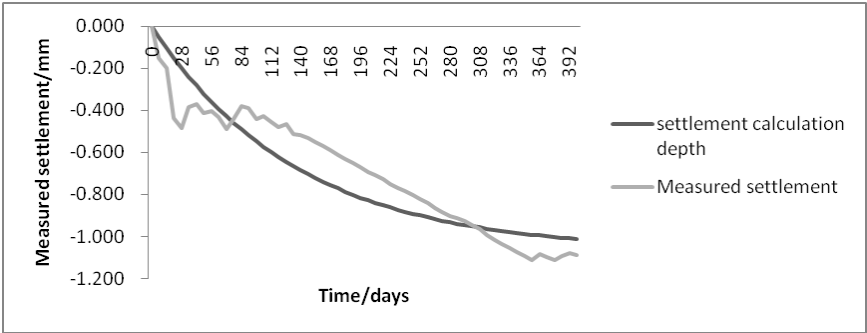


Figure 2. Measuring point 0030600D1 time-measured settlement curve

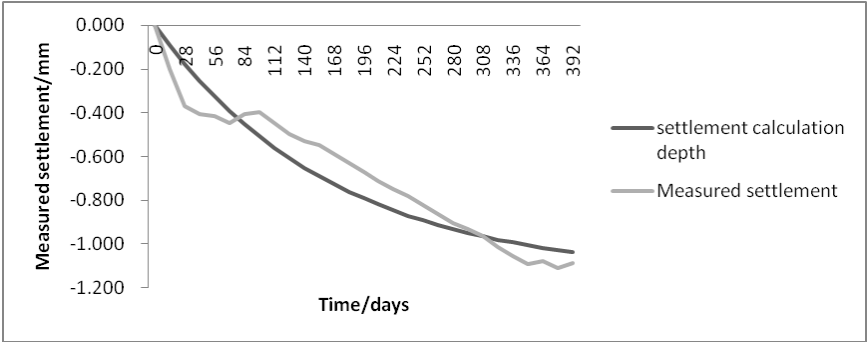


Figure 3. Measuring point 0030633D1 time-measured settlement curve

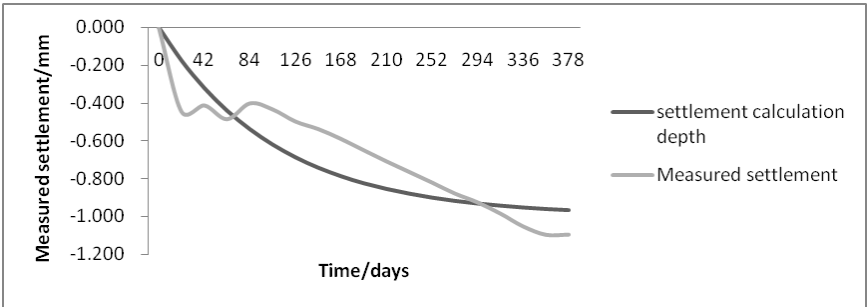


Figure 4. Measuring point 0030665D1 time-measured settlement curve

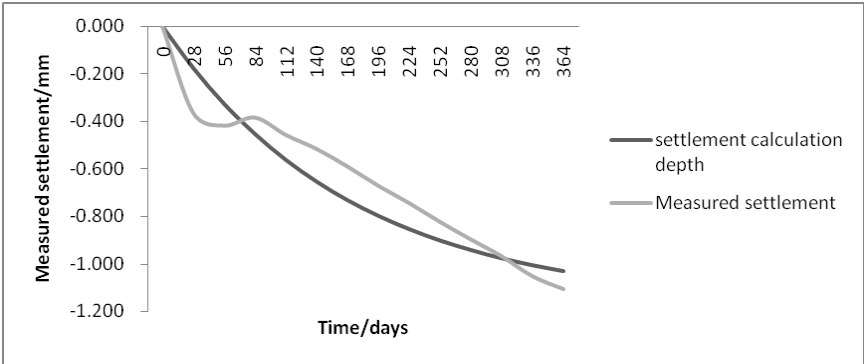


Figure 5. Measuring point 0030698D1 time-measured settlement curve

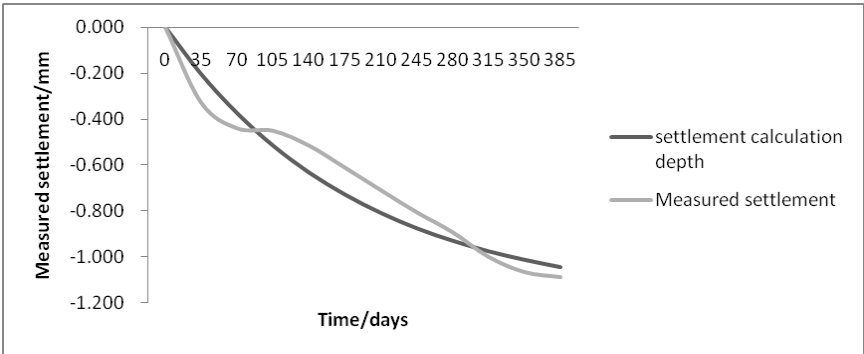


Figure 6. Measuring point 0030731D1 time-measured settlement curve

From the measured data and the data shown in Table 2 and Figures 1 to 6, in most cases, when the time interval is 21 days, the final settlement, the settlement after construction, the correlation coefficient although low, but the percentage of completion is higher; when choose 35 days, we will get higher correlation coefficient, the final settlement, the settlement after construction also larger, as the percentage of completion is lower. Although the Beijing-Shijiazhuang Passenger Line Yongding River Bridge early in the actual construction time interval chosen is seven days. But considering and analyzing various parameters, considering the construction problems in actual operation, the early settlement of the measured time interval, can choose to 21-35 days, which means measured once a month.

CONCLUSION

The engineering practices require calculation model to be simple and practical, so the analytical method is more suitable than numerical methods. Asaoka method is easy to understand with small workload. It is suitable for the final settlement

evaluation. However, how to construct the sample data to improve its forecasting accuracy and reliability are the research priorities.

Asaoka method is strongly affected by personal factors due to the artificially selected point. In order to get better safety factor, the time starting point selection should be as late as possible. Although this may extend some of the construction period, we still should try to use the late settlement data because it is easy to identify. The latter part of the development curve is closer to the actual one from the fitting chart and forecast chart. During the early stage of construction, one month time interval can get better correlation coefficient. Such projected results will be more accurate.

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The Stability Analysis of Construction Pit When Using GFRP Bars in Shield Engineering

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ABSTRACT

Subway stations or shield pit (or shaft) are often constructed by reinforced concrete supporting structure materials. These concrete retaining structure materials must be manually destroyed before launching and receiving of shield. Therefore, some security risks may arise during manual destroying, and the structure strength may be weakened. In the supporting structure, concrete supporting structure materials have been partially substituted by glass fiber reinforced plastic bars (GFRP) in shield engineering worldwide. However, physical and mechanical properties of GFRP bars differ significantly from those of reinforcing steel. So the deformation and stability of construction pit should be considered when GFRP bar is installed in the soft-eyes instead of steel reinforcement. The stability analysis of construction pit is presented here. Through three dimensional numerical simulation and field test in Beijing subway, the deformation of GFRP concrete pit was larger than that of steel concrete pit, but the deformation of GFRP concrete pit was about 15 mm. Therefore, the GFRP concrete pit was in stability. The results are useful for further application of GFRP bars in shield engineering.

INTRODUCTION

Subway stations or shield wells are often constructed by reinforced concrete supporting structure materials, these concrete retaining structure materials must be destroyed before launching and receiving of shield. So some security risks may arise during destroying, and time limit for project may be longer, and the structure strength may be weakened. In the supporting structure, concrete supporting structure materials have been partially substituted by GFRP bars in shield engineering in several tunneling projects worldwide. Dipl.-Ing. Ben Jütte introduced some examples, such as the new North-South Line of the Amsterdam subway, the Brandenburg Gate station of the new subway line 5 in Berlin, the new North-South Line of the light rail system in Cologne, and the launch and reception shafts of the Durban Harbor Tunnel Crossing in Durban, South Africa. Liu Jun presented the current situation of research and application of GFRP in shield engineering worldwide, especially the application in Beijing; Lin Gang introduces the application of in Chengdu Metro Line1; Zhong Ming introduced the application of GFRP bars in Dongguan metro station of R2 line. The results mentioned above cannot study the stability of the shafts used for launching or reception the shield. In order to reveal the deformation pattern of the shafts using GFRP bars when

excavating, three-dimensional numerical simulation is used, and a series of in-situ measurement were carried out during the whole period of construction. The results show the shaft is in stability, and the deformation of supporting using GFRP bars is under control.

The project studied in this paper is Datun subway station of line 15 in Beijing. The foundation pit or shield shaft is depth of about 20m, the supporting of which is the type of piles and anchor cables (see Figure 1), GFRP bars zone is about the depth of 12m~18m in a pile (shown in Figure 2), the GFRP bars has the same diameter ($\Phi 25$) with the steel bar, the lapped length between the GFRP bars and steel bars is 40d (d is diameter).

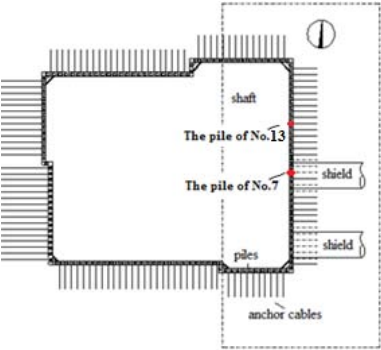


Figure 1. The supporting of foundation pit.

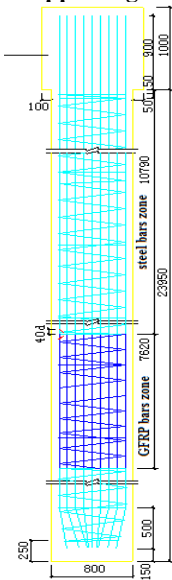


Figure 2. The position of GFRP bars zone in a pile.

NUMERICAL SIMULATION AND ANALYSIS

The FLAC3D program is performed in this calculation. The calculating model can be found in Figure 3, there are 72471 grid points and 67300 zone unit elements and 10307 structure elements in these model. The supporting structure is simulated by pile elements, the model can be found in Figure 4. In this calculation, elastoplastic model and Mohr-Coulomb yield criterion are used.

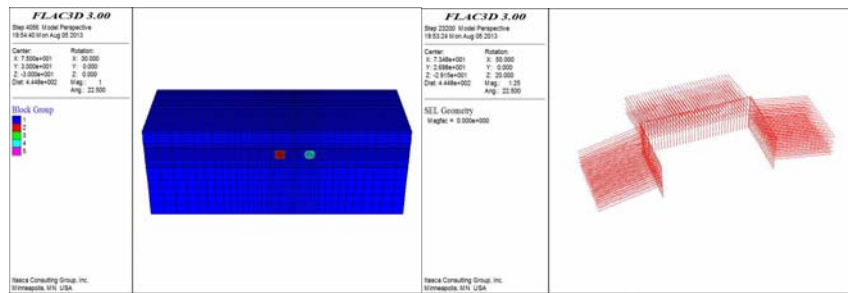


Figure 3. The calculating model. Figure 4. The supporting structure model.

According to the site exploration, the layers of this range are composed by artificial soil and clay soil and fine sand, the physical and mechanical parameters of each layer used in the calculating model are listed in Table 1.

Table 1. The calculating parameters used in calculation

Material	γ (kN/m^3)	E (MPa)	μ	c (kPa)	ϕ (°)
artificial soil	17	3.1	0.35	10	
clay soil	20	7.9	0.45	18	23
fine sand	21	10	0.40	0	28
concrete	25	28000	0.25		

The calculation follows the steps as the excavation of foundation pit or shaft, that is:

- Steps 1 excavate soils to the surface of 3.5m, and construct first anchor cables;
- Steps 2 excavate soils to the surface of 8.5m, and construct second anchor cables;
- Steps 3 excavate soils to the surface of 12m, and construct third anchor cables;
- Steps 4 excavate soils to the surface of 15m, and construct fourth anchor cables;
- Steps 5 excavate soils to the bottom of shaft, and construct fifth anchor cables.

In order to simplify the calculation, the pile of No.7 is discussed here only, the position of this pile can be found in Figure 1, and the type of structure of this pile is shown in Figure 2.

The result is shown in Figure 5. From this figure, with the excavation of shaft, the horizontal displacement of No.7 pile has a gradually increasing trend, increased from -1.6mm to -15.5mm. Step 1 to step 3, the horizontal displacement of No.7 pile

change little, the maximal value is about 5 mm. The excavation of step 4 to step 5, the horizontal displacement increases gradually, and the position of the maximum is -15.5mm at the depth of -14.0 to -16.0m.

In order to study the deformation of No.7 pile deeply, a contrastive analysis of GFRP bar (see Figure 2) and steel bar is performed. The result is shown in Figure 6. The position of this pile can be found in Figure 1, marked with the No.13 pile. The deformation pattern of No.13 pile is similar to that of No.7 pile, but the horizontal displacement of No.7 pile is larger than that of No.13 pile, and this phenomenon is obvious extremely at the depth of -14.0 to -16.0m; as the mentioned above, this place is the position of GFRP bar, and the maximal deference is about 2.5mm. The reason of this phenomenon may be explained as: in contrast with steel bars, elastic modulus of GFRP bars is very low, and ductility of GFRP bars is poor, the damage of GFRP bars has not any substantial yield deformation, which shows brittle fracture only.

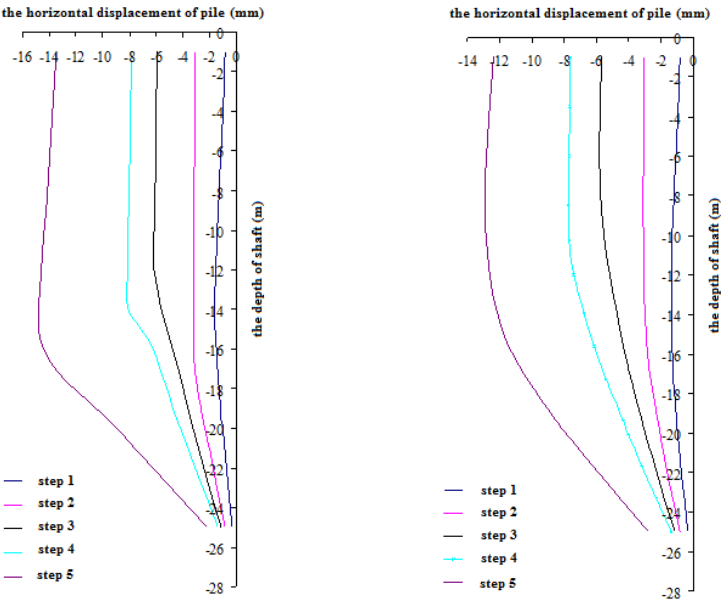


Figure 5. The deformation of GFRP pile. Figure 6. The deformation of steel pile.

Through the study of another project, the same phenomenon is revealed, which is shown in Figure 7. The position of GFRP bars is at the depth of -17.0 to -23.0m, the deformation of GFRP pile is obviously larger than that of steel pile, and the maximal deference value of deformation is about 5 mm.

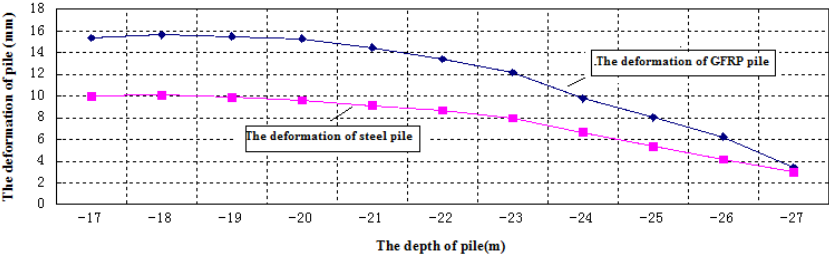


Figure 7. The deformation between steel pile and GFRP pile.

The results mentioned above show that the deformation pattern of GFRP bars pile and steel pile is similar, and the deformation of GFRP bar pile is larger than that of steel pile, particularly at the position of hole of shield, where the GFRP bars have been used for replacing the original steel bars. However, the horizontal displacement of GFRP bar pile is still smaller than 30 mm, which is the allowed maximal horizontal displacement required by “Technical Code for Monitoring Measurement of Subway Engineering” in Beijing, in other words, the stability of this shaft is in stability when using the GFRP bars.

IN-SITU MEASUREMENT

In order to ensure the horizontal displacement of No.7 pile within the prescribed limit, a series of in-situ measurement were taken out during the period of excavation.

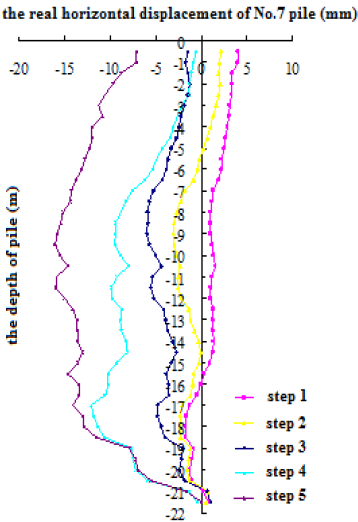


Figure 8. The real horizontal displacement of No.7 pile.

From Figure 8, the measured value is basically similar to the theoretical value, but the measured value is deferent to the theoretical value at step 1 to step 3, the reason of which is perhaps the overloaded prestressing anchor cables, however the maximal horizontal displacement is about 15 mm, which is same to the theoretical analysis, the central issue is that the real deformation is not out of the allowed maximal value. In addition, the measured surface settlement is about 5 mm, and the maximum is only 8 mm, which is far smaller than the allowed (30 mm).

CONCLUSIONS

Through installation of GFRP bars in the soft-eyes instead of steel reinforcement, non-obstruction launching and receiving of shield are successfully achieved. This method avoids the project accident and accelerates the speed of construction. Through three dimensional numerical simulation and field test in Beijing subway line 15, the deformation pattern of GFRP concrete pile was similar to that of steel concrete pile, while the deformation of GFRP concrete pile was little larger than that of steel concrete pile at the depth of soft-eyes. However, the deformation of two type piles is not beyond the allowed value which is required by Beijing code of subway construction, that is to say, the shaft using GFRP concrete pile is in stability. This point is further proved by the measured surface settlement. The results are useful for further application of GFRP rebar in shield engineering.

ACKNOWLEDGMENTS

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Analytical Solutions of Rectangular Bottom Slab of Box-Type Underground Structure under Vertical Seismic Action

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ABSTRACT

In the last years many attentions have been paid to the dynamic responses and failure of roof slab of underground box-type structures, but less attention has been paid to the dynamic responses and failure of bottom slab of underground structures. The seismic damage to bottom slabs should not be neglected, because under seismic actions bottom slabs may fracture and bulge, and produce adverse effects on the other parts of the structure. Therefore the seismic response of bottom slabs should be considered in aseismic design of underground structures. In this paper dynamic response of rectangular bottom slab of box-type underground structure under vertical seismic action was studied. In view of the complexity of dynamic soil-underground structure interaction analysis, in order to simplify the analysis we consider the roof slab and side wall as rigid ones, and only the deformation of bottom slab was considered. Hinged and clamped connections between bottom slab and side walls were considered. The eigenfunctions of motion equation of bottom slab were obtained with consideration of the restraints of the side walls on bottom slab, and the analytical solution of forced dynamic response of bottom slab under vertical seismic action was obtained.

INTRODUCTION

One issue in earthquake engineering that was dismissed in the early stage was the vertical component of earthquake ground motion (*CRIEPI Report, No. U04003*, 2004). Two reasons are thought to have resulted in this situation; doubts regarding the existence of high vertical motion and about its structural effects. With regard to the former, clearly the international strong-motion data bank was inevitably dominated by far field events with very few recordings obtained in the vicinity of sizable earthquake. Since vertical motion attenuates faster than its horizontal counterpart, it follows that regression studies will lead to relationships indicating very low peak vertical accelerations. As to structural effects, the observation that structures are designed with a high safety factor against vertical (gravity) forces leads to the conclusion that vertical earthquake-induced forces are structurally insignificant. Only in the last thirty years has the established view on vertical earthquake motion been

reinvestigated from an engineering seismology as well as earthquake engineering stand points. One of the earliest studies on peak vertical and horizontal ground motion was that undertaken by Newmark and his co-workers (Newmark et al. 1973), which lead to the conclusion that the peak vertical acceleration could be safely taken as 2/3 of the horizontal value. This value has been readily accepted by all seismic codes world wide.

In Kobe earthquake in 1995 the value of the ratio of vertical-to-horizontal components reached 1.69, with peak vertical accelerations of 0.57g (Kobe Port Island array) (Elnashai 1997). Vertical motion has been blamed for some of the extensive damage in Kobe earthquake. A steel column in the mega truss structure of the Ashiyahama cracked along a strictly horizontal plane that is most likely attributed to tensile force from combined overturning and vertical response.

Because of the failure of roof slabs in subway station Dakai in Kobe earthquake in 1995, in the last fifteen years much attention has been paid to the dynamic response and failure mechanism of roof slabs of subway stations, for instance (Uenishi and Sakurai 2000, Samata et al. 1997, Tiwatate et al. 2000, Guo et al. 2002, Zhuang et al. 2008), and less attention has been paid to the seismic response of bottom slabs. The seismic damage to bottom slabs should not be neglected, because under the seismic action the bottom slab may fracture and bulge, and produce adverse effects on the other parts of the structure. Therefore the seismic response of bottom slab should be considered in aseismic design, particularly under the vertical seismic action, because for bottom slabs the vertical seismic component has significant effect. Here for simplicity of analysis we take roof slab and side walls as rigid ones, only the deflection of bottom slab is considered. Hinged and clamped connections between bottom slab and side walls are considered. The interaction between soil and structure is modeled by Winkler-type springs. The result will serve as reference for nonlinear soil-structure interaction.

THE ANALYTICAL SOLUTION OF BOTTOM SLAB UNDER VERTICAL SEISMIC ACTION

The absolute and relative displacements of bottom slab are expressed as follows

$$w = w(x, y, y), \tilde{w} = \tilde{w}(x, y, y) \quad (1)$$

The relationship between relative displacements of bottom slab $\tilde{w}$ and the absolute displacement of soil u_0 is

$$\tilde{w}(x, y, t) = w(x, y, t) - u_0(t) \quad (2)$$

The differential equation of motion of bottom slab is ^[9]

$$\frac{\partial^4 \tilde{w}}{\partial x^4} + 2 \frac{\partial^4 \tilde{w}}{\partial x^2 \partial y^2} + \frac{\partial^4 \tilde{w}}{\partial y^4} + M_{pl} \frac{\partial^2 \tilde{w}}{\partial t^2} + k_{p0} \tilde{w} = -M_{pl} \frac{\partial^2 u_0}{\partial t^2} \quad (3)$$

Where $M_{pl} = \rho h / D$, $k_{p0} = k_p / D$, ρ is the density of bottom slab, k_p is coefficient of subgrade reaction of soil, and $Eh^3 / [12(1 - \nu^2)]$ is the flexural rigidity of bottom slab.

Suppose that in y direction at the ends of slab $y = 0$ and $y = b$ bottom slab is connected with side walls. The connection may be simply supported

$$\begin{aligned} y = 0 : \tilde{w}(x, 0, t) = 0, \quad \left[\partial^2 \tilde{w} / \partial x^2 + \nu \partial^2 \tilde{w} / \partial y^2 \right]_{y=0} &= 0 \\ y = b : \tilde{w}(x, 0, t) = 0, \quad \left[\partial^2 \tilde{w} / \partial x^2 + \nu \partial^2 \tilde{w} / \partial y^2 \right]_{y=b} &= 0 \end{aligned} \quad (4)$$

or fixed:

$$\begin{aligned} y = 0 : \tilde{w}(x, 0, t) = 0, \quad \partial \tilde{w} / \partial y|_{y=0} &= 0 \\ y = b : \tilde{w}(x, 0, t) = 0, \quad \partial \tilde{w} / \partial y|_{y=b} &= 0 \end{aligned} \quad (5)$$

In x direction the boundary conditions may be written as:

$$\begin{aligned} x = 0 : \tilde{w}(0, y, t) = \sum_{m=1}^{\infty} \tilde{u}_m \cdot \sin[(m\pi/b)y], \quad \partial w(x, y, t) / \partial x|_{x=0} &= 0 \\ x = a : \tilde{w}(a, y, t) = \sum_{m=1}^{\infty} \tilde{u}_m \cdot \sin[(m\pi/b)y], \quad \partial w(x, y, t) / \partial x|_{x=a} &= 0 \end{aligned} \quad (6)$$

Where

$$\tilde{u}_m(t) = \tilde{u}(t) \int_0^b \sin[(m\pi/b)y] dy / \int_0^b \sin^2[(m\pi/b)y] dy$$

It is clear from boundary conditions (5) and (6) that, in terms of variable x the eigenfunctions can not be determined, because the boundary condition in x direction is not homogeneous. But along y axis eigenfunctions can be determined because of the homogeneity of the boundary conditions along y axis. Therefore relative displacement may be written as (Asmar 2005)

$$\tilde{w}(x, y, t) = \bar{w}(y) \cdot f(x, t) \quad (7)$$

$\bar{w}(y)$ may be written in terms of sine functions in accordance with conditions (4):

$$\bar{w}(y) = \sin[(m\pi/b)y] \quad (8)$$

The corresponding relative displacement of bottom slab may be expressed as follows

$$\tilde{w}(x, y, t) = \sum_{m=1}^{\infty} f(x, t) \cdot \sin[(m\pi/b)y] \quad (9)$$

Where function $f(x, t)$ satisfies the following equation

$$\frac{\partial^4 f(x, t)}{\partial x^4} - \left(\frac{2m^2\pi^2}{b^2} \right) \frac{\partial^2 f(x, t)}{\partial x^2} + M_{pl} \frac{\partial^2 f(x, t)}{\partial t^2} + \left(\frac{m^4\pi^4}{b^4} + k_{p0} \right) f(x, t) = -M_{pl} \frac{\partial^2 u_{0m}}{\partial t^2} \quad (10)$$

Eq. (10) is similar to the motion equation of beam.

From Eq. (10) it is clear that $\ddot{u}_{0m}$ should have the following form

$$\ddot{u}_{0m}(t) = \ddot{u}_0(t) \frac{2}{\pi m} (1 - \cos m\pi) \quad (11)$$

Where $m = 2n + 1$.

Applying Laplacian transformation converts Eq. (10) into ordinary differential equation. Multiplying Eq. (10) by e^{-zt} and integrating over the interval $t \in [0, \infty]$ leads to

$$q^{IV}(x,z)-\left(\frac{2m^2\pi^2}{b^2}\right)q''(x,z)+\left(M_{pl}z^2+\frac{m^4\pi^4}{b^4}+k_{p0}\right)q(x,z)=-M_{pl}\tilde{u}_{0m}(z) \tag{12}$$

Solving Eq. (12) and carrying out inverse Laplacian transformation we can obtain the original function of $q : f = f(x,t)$.

By introducing the following parameters

$$\frac{m^2\pi^2}{b^2}=a^2, M_{pl}z^2+\frac{m^4\pi^4}{b^4}+k_{p0}=-\alpha \tag{13}$$

We obtain the homogeneous equation of Eq. (12)

$$q^{IV}(x,z)-2a^2q''(x,z)-\alpha q(x,z)=0 \tag{14}$$

The eigenroots of Eq. (14) are

$$\lambda_{1,2}=\pm\sqrt{a^2+\sqrt{a^4+\alpha}}, \lambda_{3,4}=\pm\sqrt{a^2-\sqrt{a^4+\alpha}} \tag{15}$$

It is necessary to distinguish real roots and imaginary roots of Eq. (14). Several cases need to be studied:

Case 1. The first two roots are real, and the last two roots are imaginary

$$\begin{aligned} \lambda_{1,2} &= \pm\sqrt{a^2+\sqrt{a^4+\alpha}} = \pm\Pi \\ \lambda_{3,4} &= \pm\sqrt{a^2-\sqrt{a^4+\alpha}} = \pm ip \end{aligned} \tag{16}$$

Case 2. All roots are real

$$\lambda_{1,2}=\pm\lambda, \lambda_{3,4}=\pm M \tag{17}$$

Case 3. All roots are imaginary

$$\begin{aligned} \lambda_{1,2} &= \pm\sqrt{a^2+\sqrt{a^4+\alpha}} \\ \lambda_{3,4} &= \pm\sqrt{a^2-\sqrt{a^4+\alpha}} \end{aligned} \tag{18}$$

Case 4. Two roots are real, and the other two roots are zero

$$\lambda_{1,2}=\pm a\sqrt{2}, \lambda_{3,4}=\pm 0 \tag{19}$$

The solutions of Eq. (12) corresponding to cases 1, 2 and 4 are as follows, respectively

$$q(x,z)=c_1e^{\lambda_1x}+c_2e^{-\lambda_1x}+c_3\cos\lambda_2x+c_4\sin\lambda_2x-M_{pl}\tilde{u}_{0m}(z)/\alpha \tag{20}$$

$$q(x,z)=c_1e^{\lambda_1x}+c_2e^{-\lambda_1x}+c_3e^{\lambda_2x}+c_4e^{-\lambda_2x}-M_{pl}\tilde{u}_{0m}(z)/\alpha \tag{21}$$

$$q(x,z)=c_1e^{\lambda_1x}+c_2e^{-\lambda_1x}+c_3+c_4x-M_{pl}\tilde{u}_{0m}(z)/\alpha \tag{22}$$

It is clear from Eq. (23) that the expression in square root symbol is complex; therefore it is impossible to write the solutions of Eq. (12). It is necessary to transform Eq. (23). When $\alpha < -a^4$ we have

$$\sqrt{a^4+\alpha}=ic \tag{23}$$

Where $c=\sqrt{-(a^4+\alpha)}$. The corresponding eight roots are as follows

Now let's write the solutions corresponding to the following roots in the third case

$$\begin{aligned}\lambda_{1,2} &= \pm\sqrt{a^2 + \sqrt{a^4 + \alpha}} = \pm\sqrt{a^2 + ic} \\ \lambda_{3,4} &= \pm\sqrt{a^2 - \sqrt{a^4 + \alpha}} = \pm\sqrt{a^2 - ic}\end{aligned}\quad (24)$$

$$\begin{aligned}\lambda_{10} &= \sqrt{r}[\cos[\arctg(c/a^2)] + i\sin[\arctg(c/a^2)]] \\ \lambda_{11} &= \sqrt{r}\{\cos[2\arctg(c/a^2) + 2\pi]/2 + i\sin[2\arctg(c/a^2) + 2\pi]/2\}, \\ \lambda_{20} &= -\sqrt{r}[\cos\arctg(c/a^2) + i\sin\arctg(c/a^2)], \\ \lambda_{21} &= \sqrt{r}\{\cos[2\arctg(c/a^2) + 2\pi]/2 + i\sin[2\arctg(c/a^2) + 2\pi]/2\}, \\ \lambda_{30} &= \sqrt{r}\{\cos\arctg(c/a^2) - i\sin[2\arctg(c/a^2) - 2\pi]/2\}, \\ \lambda_{31} &= \sqrt{r}\{\cos[2\arctg(c/a^2) - 2\pi]/2 - i\sin[2\arctg(c/a^2) - 2\pi]/2\}, \\ \lambda_{40} &= -\sqrt{r}[\cos\arctg(c/a^2) - i\sin\arctg(c/a^2)], \\ \lambda_{41} &= -\sqrt{r}\{\cos[2\arctg(c/a^2) - 2\pi]/2 - i\sin[2\arctg(c/a^2) - 2\pi]/2\}\end{aligned}\quad (25)$$

Eq. (25) may be expressed as

$$\lambda_{1,2} = A \pm iB, \lambda_{3,4} = -A \pm iB \quad (26)$$

Where

$$A = \sqrt{r} \cdot \cos[\arctg(c/a^2)], B = \sqrt{r} \cdot \sin[\arctg(c/a^2)]$$

Therefore when $\alpha < -a^4$ the solution of nonhomogeneous differential equation has the following form

$$q(x, z) = c_1 e^{Ax} \cos Bx + c_2 e^{Ax} \sin Bx + c_3 e^{-Ax} \cos Bx + c_4 e^{-Ax} \sin Bx - M_{pi} \tilde{u}_{0m}(z)/\alpha \quad (27)$$

For boundary conditions by taking Fourier transform in x we obtain

$$\begin{aligned}x = 0, q(0, z) &= \tilde{u}(z), \partial q(x, z)/\partial x|_{x=0} = 0 \\ x = a, q(a, z) &= \tilde{u}(z), \partial q(x, z)/\partial x|_{x=a} = 0\end{aligned}\quad (28)$$

Substituting Eqs. (20), (21), (22), (25) into Eq. (28), respectively, we obtain 4 algebraic equations for determining the integral constants. After determination of these integral constants we obtain

$$q(x, z) = (\Delta_1/\Delta)k_1 + (\Delta_2/\Delta)k_2 + (\Delta_3/\Delta)k_3 + (\Delta_4/\Delta)k_4 - M_{pi} \tilde{u}_{0m}(z)/\alpha \quad (29)$$

Where $k_1 \sim k_4$ represent

$$\begin{aligned}k_1 &: e^{\lambda_1 x}, e^{\lambda_1 x}, e^{\lambda_1 x}, e^{Ax} \cos Bx; \\ k_2 &: e^{-\lambda_1 x}, e^{-\lambda_1 x}, e^{-\lambda_1 x}, e^{Ax} \sin Bx; \\ k_3 &: \cos \lambda_2 x, e^{\lambda_2 x}, 1, e^{-Ax} \cos Bx; \\ k_4 &: \sin \lambda_2 x, e^{-\lambda_2 x}, x, e^{-Ax} \sin Bx.\end{aligned}\quad (30)$$

The frequency equations of bottom slab may be obtained by equating the determinants to zero in the cases 1, 2 and 3

$$\begin{aligned}\Delta(\alpha) &= 4\lambda_1 \lambda_2 - 2\lambda_1 \lambda_2 \cos \lambda_2 a \cdot (e^{\lambda_1 a} + e^{-\lambda_1 a}) - \\ &- \lambda_2^2 \sin \lambda_2 a \cdot (e^{\lambda_1 a} + e^{-\lambda_1 a}) + \lambda_1^2 \sin \lambda_2 a \cdot (e^{\lambda_1 a} - e^{-\lambda_1 a}) = 0\end{aligned}\quad (31)$$

$$\Delta(\alpha) = 2A^2 \sin^2 Ba + B^2 - B^2 e^{-2Aa} = 0 \quad (32)$$

$$\Delta(\alpha) = ch[(\lambda_1 + \lambda_2)a] - [(\lambda_1 + \lambda_2)^2 / (\lambda_2 - \lambda_1)^2] ch[(\lambda_1 - \lambda_2)a] + [4\lambda_1\lambda_2 / (\lambda_2 - \lambda_1)^2] = 0 \quad (33)$$

Their solutions are

$$\alpha = \alpha_k(m), \quad m=1,2,3, \dots \quad (34)$$

The determinant of algebraic equations, the Laplacian transformation of the displacement of the structure as a rigid one and the acceleration of soil enter the solution of Eq. (29)

$$\tilde{u}(z) + M_{pl}\tilde{u}_{0m}/\alpha = T \quad (35)$$

Now we derive Eq. (35) from Δ_1 , Δ_2 , Δ_3 , and Δ_4 . We introduce the following designation

$$\Delta_1 = \bar{\Delta}_1 T, \Delta_2 = \bar{\Delta}_2 T, \Delta_3 = \bar{\Delta}_3 T, \Delta_4 = \bar{\Delta}_4 T \quad (36)$$

Consider that Eq. (29) has 4 particular solutions

$$k_1(x), k_2(x), k_3(x), k_4(x) \quad (37)$$

Substituting Eqs. (36) and (37) into Eq. (29) leads to

$$q(x, z) = T(p) \left[(\bar{\Delta}_1 / \Delta) k_1 + (\bar{\Delta}_2 / \Delta) k_2 + (\bar{\Delta}_3 / \Delta) k_3 + (\bar{\Delta}_4 / \Delta) k_4 \right] - M_{pl} \tilde{u}_{0m}(z) / \alpha \quad (38)$$

As we know, $\tilde{u}(z)$ is the Laplacian transformation of function $\tilde{u}(t)$

$$\tilde{u}(t) = \frac{1}{p} \int_0^t \ddot{u}_0(\tau) \sin p(t - \tau) d\tau \quad (39)$$

From theorem of convolution we have

$$\tilde{u}(z) = \ddot{u}_0(z) p / (p^2 + z^2) \quad (40)$$

Therefore Eq. (41)

$$\ddot{u}_{0m}(z) = \ddot{u}_0(z) \frac{2}{\pi m} (1 - \cos m\pi) \quad (41)$$

is the Laplacian of the following function

$$\ddot{u}_{0m}(t) = \ddot{u}_0(t) \frac{2}{\pi m} (1 - \cos m\pi) \quad (42)$$

Substituting Eq. (40) and (41) into Eq. (35) gives

$$\begin{aligned} T &= \tilde{u}(z) + M_{pl} \ddot{u}_{0m}(z) / \alpha \\ &= \ddot{u}_0(z) \cdot [p / (p^2 + z^2) + 2M_{pl} (1 - \cos m\pi) / (\alpha m \pi)] \end{aligned} \quad (43)$$

With the help of Eq. (43) and (41), Eq. (38) may be rewritten as follows

$$\begin{aligned} q(x, z) &= \ddot{u}_0(z) \cdot [p / (p^2 + z^2) + 2M_{pl} (1 - \cos m\pi) / (\alpha m \pi)] \cdot \\ &\quad [(\bar{\Delta}_1 / \Delta) k_1 + (\bar{\Delta}_2 / \Delta) k_2 + (\bar{\Delta}_3 / \Delta) k_3 + (\bar{\Delta}_4 / \Delta) k_4] - \\ &\quad \ddot{u}_0(z) \frac{2M_{pl}}{\alpha \pi m} (1 - \cos m\pi) \end{aligned} \quad (44)$$

From the expression of α in Eq. (13)

$$M_{pl} z^2 + \frac{m^4 \pi^4}{b^4} + k_{p0} = -\alpha \quad (45)$$

We obtain

$$z^2 + \Lambda_m^2 = -\alpha / M_{pl} \quad (46)$$

Where

$$\Lambda_m^2 = \frac{m^4 \pi^4 / b^4 + k_{p0}}{M_{pl}} \quad (47)$$

Substituting Eq. (46) into Eq. (44), it gives

$$\begin{aligned} q(x, z) = \ddot{u}_0(z) \cdot \left[p / (p^2 + z^2) + 2M_{pl} (1 - \cos m\pi) / (\alpha m\pi) \right] \cdot \\ \left[(\bar{\Delta}_1 / \Delta) k_1 + (\bar{\Delta}_2 / \Delta) k_2 + (\bar{\Delta}_3 / \Delta) k_3 + (\bar{\Delta}_4 / \Delta) k_4 \right] \\ - \ddot{u}_0(z) \frac{2(1 - \cos m\pi)}{\pi m \Lambda_m} \cdot \frac{\Lambda_m}{(z^2 + \Lambda_m^2)} \end{aligned} \quad (48)$$

By introducing the following designations

$$\begin{aligned} p / (p^2 + z^2) + 2M_{pl} (1 - \cos m\pi) / (\alpha m\pi) = A(z, m, \alpha) \\ \frac{2(1 - \cos m\pi)}{\pi m \Lambda_m} = B(m, \Lambda_m) \end{aligned} \quad (49)$$

$$(\bar{\Delta}_1 / \Delta) k_1 + (\bar{\Delta}_2 / \Delta) k_2 + (\bar{\Delta}_3 / \Delta) k_3 + (\bar{\Delta}_4 / \Delta) k_4 = f(z) \quad (50)$$

We can rewrite Eq. (48) as follows

$$q(x, z) = A(m, z, \alpha) \ddot{u}_0(z) \cdot f(z) - B(m, \Lambda_m) \cdot \frac{\Lambda_m}{(z^2 + \Lambda_m^2)} \ddot{u}_0(z) \quad (51)$$

By using theorems of convolution and expansion we can obtain the original function $f(x, t)$

$$\begin{aligned} f(x, t) = \sum_{k=1}^{\infty} \left\{ \left[\bar{\Delta}_1^*(\alpha_k) k_1 + \bar{\Delta}_2^*(\alpha_k) k_2 + \bar{\Delta}_3^*(\alpha_k) k_3 + \bar{\Delta}_4^*(\alpha_k) k_4 \right] \cdot \right. \\ \left. / \left[\partial \Delta(\alpha) / \partial \alpha \Big|_{\alpha=\alpha_k} \partial \alpha / \partial z \Big|_{z=z_k} \right] \right\} \cdot A(m, z_k, \alpha_k) \cdot \int_0^t \ddot{u}_0(\tau) \sin(z_k(t - \tau)) d\tau - \\ - B(m, \Lambda_m) \cdot \int_0^t \ddot{u}_0(\tau) \sin(\Lambda_m(t - \tau)) d\tau \end{aligned} \quad (52)$$

Substituting Eq. (52) into Eq. 9), it gives the general solution of the motion equation of bottom slab

$$\begin{aligned} \tilde{w}(x, y, t) = \sum_{m=1}^{\infty} \sin[(m\pi/b)y] \left\{ \sum_{k=1}^{\infty} \left[\bar{\Delta}_1^*(\alpha_k) k_1 + \bar{\Delta}_2^*(\alpha_k) k_2 + \bar{\Delta}_3^*(\alpha_k) k_3 + \bar{\Delta}_4^*(\alpha_k) k_4 \right] \cdot \right. \\ \left. / \left[\partial \Delta(\alpha) / \partial \alpha \Big|_{\alpha=\alpha_k} \partial \alpha / \partial z \Big|_{z=z_k} \right] \right\} \cdot A(m, z_k, \alpha_k) \cdot \int_0^t \ddot{u}_0(\tau) \sin(z_k(t - \tau)) d\tau - \\ - B(m, \Lambda_m) \cdot \int_0^t \ddot{u}_0(\tau) \sin(\Lambda_m(t - \tau)) d\tau \end{aligned} \quad (53)$$

CONCLUSIONS

In this paper dynamic response of rectangular bottom slab of box-type underground structure under vertical seismic action was studied. In view of the complexity of dynamic soil-underground structure interaction analysis, in order to simplify the analysis we consider the roof slab and side wall as rigid ones, and only the deformation of bottom slab was considered. Hinged and clamped connections between bottom slab and side walls were considered. The eigenfunctions of motion

equation of bottom slab were obtained with consideration of the restraints of the side walls on bottom slab, and the analytical solution of forced dynamic response of bottom slab under vertical seismic action was obtained.

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Simplified Model of Seismic Soil-Underground Box-type Structure Interaction

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ABSTRACT

In this paper, a simplified soil-underground structure interaction analysis is presented. The soil was modeled as Winkler elastic ground, with which the box-type structure interacts. A matrix method was used for the soil-underground structure interaction analysis. The primary unknowns of the problem were structure joint rotations and translations. The axial deformation of the members of the structure was neglected. The rotation of structure as a whole was considered to reduce the number of the primary unknowns. The stiffness coefficients of the structure were obtained based on the theory of beam-on-elastic ground. Davis's (2003) dynamic soil pressure distribution over sidewall height was used to determine the equivalent joint loads, and a stiffness assembly method was used to obtain the governing equation of the problem. One numerical example is given to improve the effectiveness of the proposed model.

Key Words: Governing equations, simplified seismic analysis, soil-underground structure interaction, matrix method

INTRODUCTION

Since the Great Hanshin earthquake in Japan in 1995, the issue of seismic safety of underground structures has become the focus of many investigators. In the analysis of soil-underground structure interaction, two approaches are popular (St. John and Zahrah 1987; Hashash et al. 2001). One approach is the so-called complete interaction analysis, in which soil and structure are viewed as a whole, and the finite element method is used. The second approach is the simplified action-reaction analysis, in which the structure interacts with elastic or plastic ground and the ground experiences the design ground motion, also called the structural mechanics method. The complete interaction analysis is clearly superior from a theoretical viewpoint in that the complete interaction analysis comes

closest to representing all the important aspects of the problem. Its principal limitation is its complexity and the time-consuming nature. The major disadvantage of the simplified interaction analysis is that it is difficult to know by how much the results of the analysis may be conservative. The major advantage of the simplified interaction method is that it readily permits the use of approximations, which greatly reduce computational time and expense and make it convenient for engineering practice. Therefore, it was necessary to develop simplified interaction analysis methods together with the development of complete interaction analysis methods.

In this paper, a simplified soil-underground structure interaction analysis is presented. The soil is modeled as elastic ground, with which the box-type structure interacts. A matrix method was used for the soil-underground structure interaction analysis. The primary unknowns of the problem were structure joint rotations and translations. The axial deformation of the members of the structure was neglected. The rotation of structure as a whole was considered to reduce the number of the primary unknowns. The stiffness coefficients of the structure were obtained based on the theory of the beam-on-elastic ground. Davis's (2003) dynamic soil pressure distribution over sidewall was used to determine the equivalent joint loads, and a stiffness assembly method was used to obtain the governing equation of the problem. One numerical example is given to improve the effectiveness of the proposed model.

THE ANALYTICAL MODEL

Figure 1 shows the shape and dimension of the box culvert section, which consisted of sidewalls, top and bottom slabs, and intermediate columns. The column was idealized as completely fixed to the slabs. Here, soil was modeled as an isotropic elastic medium having a unit weight ρ , shear modulus G , and Poisson ratio μ . The unit-length structure has a width L and vertical walls of height h and is buried a depth d with its base at depth H below the stress-free surface. The excitation was a vertically incident shear wave propagating with a velocity C_s . To examine the peak sectional forces and moments due only to the seismic excitation, gravitation was excluded.

The seismic design of underground structures is unique in several ways. For most underground structures, the inertia of the surrounding soil is large relative to the inertia of the structure. Measurements made by Okamoto et al. (1973) of the seismic response of an immersed tube tunnel during several earthquakes showed that the response of a tunnel is dominated by the surrounding ground response and not the inertial properties of the tunnel structure itself. The focus of underground seismic design; therefore, is on the free-field deformation of the ground and its interaction with the structure. This concentration on displacement is in stark contrast to the design of above-ground structures, which focuses on inertial effects of the structure itself. This research focuses only on the determination of the stress resulting from the action of seismic wave.

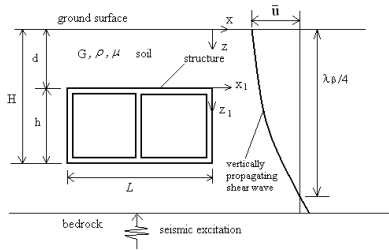


Figure 1. Underground structure analysis model.

According to Scott (1973) and Veletsos and Younan (1994), the transient stress resulting from the passage of seismic wave can be expressed as

$$\sigma_s = k_s (u_f - u_H) \quad (1)$$

Where u_f is the horizontal soil free-field displacement during seismic excitation; u_H is the horizontal soil free-field displacement at depth H ; and k_s is the soil stiffness parameter defined by the following equation (Davis, 2003):

$$k_s = \frac{2G}{\sqrt{(1-\mu)(2-\mu)}} \cdot k_\beta \cdot e^{-k_\beta x / \psi_e} = \psi_\sigma G k_\beta e^{-k_\beta x / \psi_e}, \psi_\sigma = \frac{2}{\sqrt{(1-\mu)(2-\mu)}} \quad (2)$$

In the coordinate systems defined in Figure 1, according to Davis (2003), the horizontal displacement over the region $0 \leq z \leq H$ can be expressed as

$$u = \bar{u} \left(1 - e^{-k_\beta x_1 / \psi_e} \right) \left(\cos k_\beta (z_1 + d) - \cos k_\beta H \right) e^{-i\omega t}, \psi_e^2 = (2-\mu)/(1-\mu), H \leq \lambda_\beta / 4 \quad (3)$$

Where $\bar{u} = -a_h / \omega^2$ is the magnitude of the horizontal free-field ground surface displacement; a_h is the horizontal acceleration; $k_\beta = 2\pi / \lambda_\beta$ is the shear wave number; λ_β is the shear wave length; $\omega = 2\pi f$ is the angular frequency, f is the wave frequency; and $\beta = (G/\rho)^{0.5}$ is the average free-field shear wave velocity of soil medium.

To perform soil-structure interaction analysis, it was necessary first to determine the pressure applied by the soil on the structure. For this purpose, we assumed that the bottom slab was fixed at the base and the sidewalls were fixed against rotation at the base and restrained against lateral movement.

According to Davis (2003), the horizontal compressive stress against the sidewall is determined by

$$\sigma_x = \rho a_h \frac{\psi_\sigma \beta}{2\pi f} \cdot \left(\frac{z_1 + d}{H} \right)^m \left(\cos k_\beta (z_1 + d) - \cos k_\beta H \right) e^{-i\omega t} \quad (4)$$

Where the term $((z_1 + d)/H)^m$ describes the variation of shear modulus with the depth with $0 \leq m \leq 1$. Here, we take the averaged shear modulus over the structure height. The resultant force in the horizontal central axial line is denoted by F_c , and the resultant moment about the center of the structure is M_c . It is clear that F_c will induce only horizontal translation together with soil; hence, the contribution of F_c to deformation and stressing of structure is neglected.

To simplify the analysis, the original sinusoidal lateral stress distribution was replaced by the equivalent lateral stress distribution as shown in Figure 2(b). In turn, the equivalent lateral stress distribution can be decomposed into uniform distribution and non-uniform distribution, as shown in Figures 3(b) and (c).

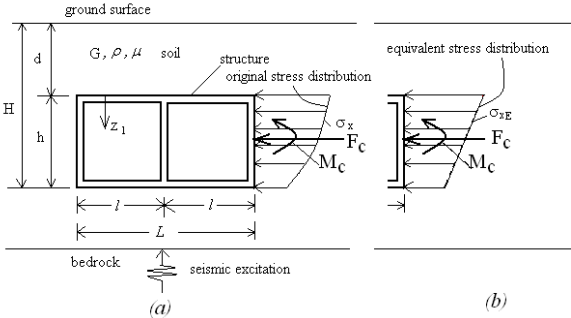


Figure 2. (a) Original lateral pressure distribution; (b) equivalent lateral pressure distribution.

GOVERNING EQUATION

The next step was to remove the restraints against the motion of the structure. It is clear that the non-uniform distribution will induce the rotation (Figure 4[a]) and racking of the structure (Figure 4[b]).

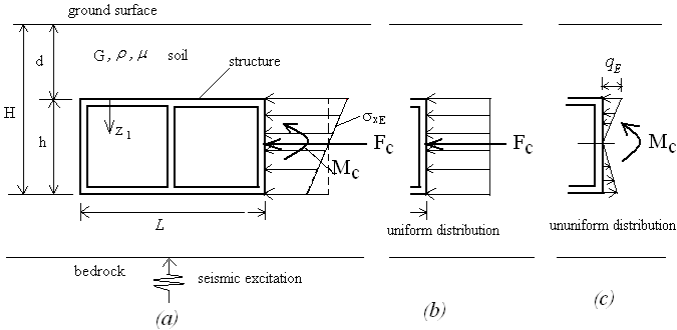


Figure 3. Decomposition of equivalent lateral pressure distribution.

We determined the reactions of the structure induced by the rotation of the structure as a whole by assuming that the rotation was rigid body rotation. In the case of infinitesimal rotation angle ϕ and the complete cohesion between soil and structure, the moment equilibrium equation may be written as

$$M_E - M_n(\phi) - M_\tau(\phi) = 0 \tag{5}$$

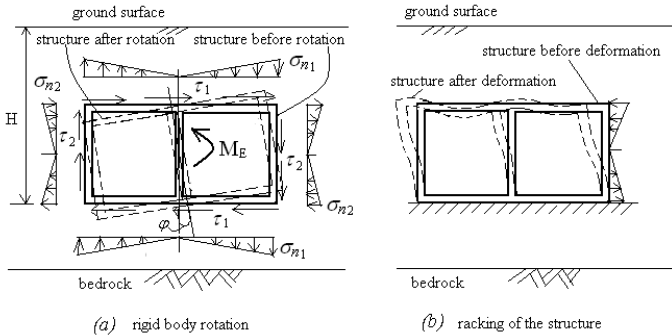


Figure 4. Motion of structure induced by non-uniformly-distributed lateral pressure in Figure 3(c).

Where M_n is the moment induced by the normal reaction forces with maximum magnitude σ_{ni} and M_τ is the moment induced by the shear reaction forces with maximum magnitude τ_i .

For the determination of rotation angle ϕ , we used the model of rigid beam on elastic foundation. Moments induced by normal reaction stresses σ_{ni} and shear reaction forces τ_i can be expressed as

$$M_n = k \left(\frac{4}{3} l^3 + \frac{1}{6} h^3 \right) \phi; M_\tau = k_\tau (h^2 l + 2 l^2 h) \phi \quad (6)$$

Where k is the elastic coefficient of ground for normal stress; k_τ is the elastic coefficient of ground for shear stress; the shear forces τ_i are supposed to be uniform at the surfaces of the structure, and $\tau_1 = k_\tau (h/2) \phi$, $\tau_2 = k_\tau l \phi$. Therefore, the rotation angle ϕ may be expressed as

$$\phi = \frac{M_E}{k \left(\frac{4}{3} l^3 + \frac{1}{6} h^3 \right) + k_\tau (h^2 l + 2 l^2 h)} \quad (7)$$

After the determination of the rotation angle ϕ , the reaction forces of the structure induced by the rotation of the structure as a whole were determined, which will affect the racking of the structure. The reaction forces are applied on the structure shown in Figure 5(b) to consider their influence on the racking of the structure. The equivalent joint forces in Figure 5(b), F_{Ei} and M_{Ei} , were determined by taking into account these reaction forces.

The numeration of the elements and joints is shown in Figure 5. The global coordinate system XOY and local coordinate systems x' are defined in Figure 5. We considered the symmetric problem only and, therefore, have only three types of elements: ① and ② are the first type, ③ and ⑤ are the second type, and ④ is the third type.

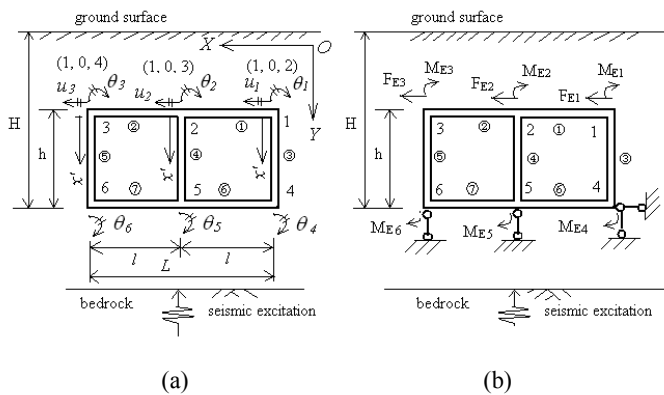


Figure 5. Analytical model for calculation of racking of structure.

The elements in the stiffness matrices of the first, second, and third types are denoted by indices 1, 2 and 3, respectively. The stiffness matrices of elements ① and ② in the global coordinate system are coincident with that in local coordinate systems and are determined by

$$[\bar{k}] = \begin{bmatrix} EA/L & 0 & 0 & -EA/L & 0 & 0 \\ 0 & J & -T & 0 & -J' & -T' \\ 0 & -T & S & 0 & T' & S' \\ -EA/L & 0 & 0 & EA/L & 0 & 0 \\ 0 & -J' & T' & 0 & J & T \\ 0 & -T' & S' & 0 & T & S \end{bmatrix} \quad (8)$$

where l is the length of element; E is Young's elastic modulus; A is the section area of elements; and J, T, S, J', T' are stiffness coefficients of elements on elastic foundation (Long, 1981).

For elements ③, ④, and ⑤, after coordinate transformation, their stiffness can be expressed as

$$[\bar{k}] = \begin{bmatrix} J & 0 & T & -J' & 0 & T' \\ 0 & EA/L & 0 & 0 & -EA/L & 0 \\ T & 0 & S & -T' & 0 & S' \\ -J' & 0 & -T' & J & 0 & -T \\ 0 & -EA/L & 0 & 0 & EA/L & 0 \\ T' & 0 & S' & -T & 0 & S \end{bmatrix} \quad (9)$$

If we neglect the axial deformation of the elements, and the horizontal displacement of bottom slab is considered to be zero, then we have

$$u_1 = u_2 = u_3 = u \quad (10)$$

Using the stiffness assembly method, we obtained the global stiffness matrix of the whole structure and furthered the governing equation of the problem as follows.

$$\begin{Bmatrix} F_{E1} + F_{E2} + F_{E3} \\ M_{E1} \\ M_{E2} \\ M_{E3} \\ M_{E4} \\ M_{E5} \\ M_{E6} \end{Bmatrix} = \begin{bmatrix} J_3 + 12EI_4/h^3 + J_4 & T_3 & 6EI_4/h^2 & T_5 & T_3' & 6EI_4/h^2 & T_5' \\ T_3 & S_1 + S_3 & S_1' & 0 & S_3' & 0 & 0 \\ 6EI_4/h^2 & S_1' & S_1 + S_2 + 4EI_4/h & S_2' & 0 & 2EI_4/h & 0 \\ T_3 & 0 & S_2' & S_2 + S_5 & 0 & 0 & 0 \\ T_3' & S_3' & 0 & 0 & S_3 + S_6 & S_6' & 0 \\ 6EI_4/h^2 & 0 & 2EI_4/h & 0 & S_6' & S_6 + S_7 + 4EI_4/h & S_7' \\ T_5' & 0 & 0 & S_5' & 0 & S_7' & S_5 + S_7 \end{bmatrix} \begin{Bmatrix} u \\ \theta_1 \\ \theta_2 \\ \theta_3 \\ \theta_4 \\ \theta_5 \\ \theta_6 \end{Bmatrix} \quad (11)$$

Where parameters J_4 and T_4 for the intermediate column are calculated by $J_4 = 12EI/h^3$, $T_4 = 6EI/h^2$, $S_3 = 4EI/h$, where EI is the flexural rigidity of intermediate column. The other stiffness coefficients in Eq. (11) were calculated according to Long (1981).

The most unfavorable lateral pressure distribution for racking should be the one by which the pressure at the base of the structure is zero. We selected this type of lateral pressure distribution for calculation. According to structural mechanics, the equivalent joint loads are determined as follows.

$$\begin{aligned} F_{E1} + F_{E2} + F_{E3} &= \frac{7}{10} q_E h - \frac{2}{5} \sigma_{n2} h - 2\tau_1 l; M_{E1} = \frac{1}{20} q_E h^2 - \frac{1}{60} \sigma_{n2} h^2 - \frac{1}{20} \sigma_{n1} l^2 \\ M_{E4} &= -\frac{1}{30} q_E h^2 - \frac{1}{60} \sigma_{n2} h^2 - \frac{1}{20} \sigma_{n1} l^2; M_{E2} = \frac{1}{15} \sigma_{n1} l^2 = M_{E5} \\ M_{E3} &= \frac{1}{20} q_E h^2 - \frac{4\sigma_{n1} l^2 + \sigma_{n2} h^2}{80}; M_{E6} = -\frac{1}{30} q_E h^2 - \frac{4\sigma_{n1} l^2 + \sigma_{n2} h^2}{80} \end{aligned} \quad (12)$$

To prove the correctness of the model, one numerical example is given. The unit-length along axis monolithic structure has width $L = 2 \times 7 = 14m$ and vertical wall height $h = 6m$; the burial depth is $d = 15m$ and the depth of base is $H = 21m$. The soil has a density $\rho_s = 2000kg/m^3$, Poisson's ratio $\mu_s = 0.3$, shear wave propagation velocity $v_s = 150m/s$, and Young's modulus $G_s = 4.5 \times 10^7 N/m^2$. Young's shear modulus of concrete is supposed to be $G_c = 2.5 \times 10^{10} N/m^2$. The horizontal component of seismic excitation has a peak acceleration $\alpha_h = 5.78m/s^2$, frequency $f = 2$ Hertz ($\omega = 2\pi f = 12.57$). The other parameters for computation are $\lambda_\beta = TV_s = 75m$, $k_\beta = 2\pi/\lambda_\beta = 0.084$, $\psi_\sigma = 2/[(1-\mu)(2-\mu)]^{0.5} = 1.8334$. The distribution of loads induced by horizontal seismic component is represented in Figure 6.

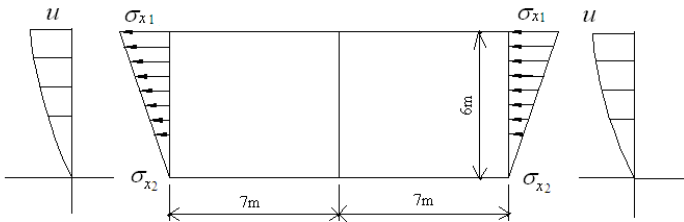


Figure 6. Distribution of loads induced by horizontal seismic component.

After calculation, we obtained the governing equation of the problem as follows:

$$\begin{Bmatrix} 443666.678 \\ 158598269 \\ 83601.18 \\ 160243.48 \\ -221201.73 \\ 83601.18 \\ -219556.518 \end{Bmatrix} = \begin{bmatrix} 6.125 \times 10^8 & 2.27 \times 10^8 & 4.34 \times 10^7 & 2.27 \times 10^8 & 7.10 \times 10^6 & 4.34 \times 10^7 & 7.10 \times 10^6 \\ 2.27 \times 10^8 & 6.88 \times 10^8 & -2.20 \times 10^6 & 0 & 6.58 \times 10^6 & 0 & 0 \\ 4.34 \times 10^7 & -2.20 \times 10^6 & 8.62 \times 10^8 & -2.20 \times 10^6 & 0 & 8.68 \times 10^7 & 0 \\ 2.27 \times 10^8 & 0 & -2.20 \times 10^6 & 6.88 \times 10^8 & 0 & 0 & 6.58 \times 10^6 \\ 7.10 \times 10^6 & 6.58 \times 10^6 & 0 & 0 & 6.88 \times 10^8 & -2.20 \times 10^6 & 0 \\ 4.34 \times 10^7 & 0 & 8.68 \times 10^7 & 0 & -2.20 \times 10^6 & 8.62 \times 10^8 & -2.20 \times 10^6 \\ 7.10 \times 10^6 & 0 & 0 & 6.58 \times 10^6 & 0 & -2.20 \times 10^6 & 6.88 \times 10^8 \end{bmatrix} \begin{Bmatrix} u \\ \theta_1 \\ \theta_2 \\ \theta_3 \\ \theta_4 \\ \theta_5 \\ \theta_6 \end{Bmatrix}$$

The solution of the governing equation is:

$$\begin{Bmatrix} u \\ \theta_1 \\ \theta_2 \\ \theta_3 \\ \theta_4 \\ \theta_5 \\ \theta_6 \end{Bmatrix} = \begin{Bmatrix} 7.28 \times 10^{-4} \\ -6.41 \times 10^{-6} \\ 5.49 \times 10^{-5} \\ -4.04 \times 10^{-6} \\ -3.29 \times 10^{-4} \\ 5.31 \times 10^{-5} \\ -3.26 \times 10^{-4} \end{Bmatrix}$$

The final bending moment diagram of the structure is demonstrated in Figure 7. The consideration of inertial effect is left for future work.

CONCLUSION

In this paper, a simplified soil-underground structure interaction analysis is presented. The soil was modeled as Winkler elastic ground, with which the box-type structure interacts. The primary unknowns of the problem were structure joint rotations and translations. The axial deformation of the members of the structure was neglected, and the rotation of structure as a whole was considered. In this way, the number of the primary unknowns was reduced, and the stiffness coefficients of the structure were obtained based on the theory of the beam-on-elastic ground. Davis’s (2003) dynamic pressure distribution over sidewall was used to determine the equivalent joint loads. Finally, the governing equation of the problem was obtained.

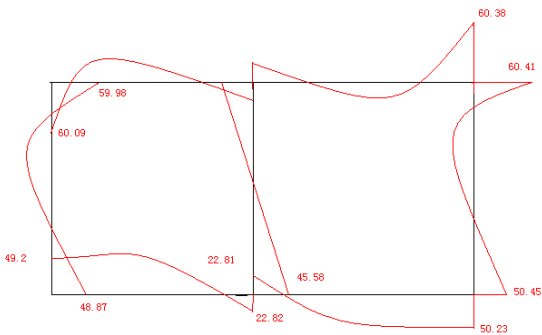


Figure 7. Bending moment diagram of structure (kN · m).

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