Environmental Sustainability in Transportation Infrastructure

Selected Papers from the International Symposium on Systematic Approaches to Environmental Sustainability in Transportation

August 2–5, 2015
Fairbanks, Alaska

Edited by

Jenny Liu, Ph.D., P.E.; Sheng Zhao, Ph.D.; and Peng Li, Ph.D.

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Preface

Environmental stewardship is an important factor to consider during the construction, operations, and preservations of transportation systems. This is particularly true with the more stringent environmental regulations recently developed. *Environmental Sustainability for Transportation Infrastructure* Special Technical Publication (STP) selects 22 papers that represent recent development, practices, and advances to maximize environmental sustainability of transportation infrastructure. These papers cover a wide array of topics under three groups: 1) Managing stormwater runoff through improved monitoring, advanced technology, and pervious concrete (four papers); 2) Reducing environmental impacts during construction, operations, and preservation (10 papers); and 3) Life cycle costing and assessment, energy consumption, and environmental assessment (eight papers).

Two or more reviewers along with the editors evaluated each paper published in this ASCE STP. The authors of the accepted papers have addressed all the reviewers' comments to the satisfaction of the editors. All published papers are eligible for discussion in the *Journal of Materials in Civil Engineering*, and are eligible for ASCE awards.

The papers collected in this publication were presented at the *International Symposium on Systematic Approaches to Environmental Sustainability in Transportation* held in Fairbanks, Alaska, USA from August 2 to 5, 2015. This conference was hosted by the Center for Environmentally Sustainable Transportation in Cold Climates (CESTiCC), Chinese Society of Civil Engineers, and Tongji University, China in collaboration with Environmental UTC Network, International Association of Chinese Infrastructure Professionals, the Infrastructure & Climate Network (ICNet), University of Alaska Fairbanks in USA, University of Kansas in USA, University of Tennessee in USA, and Wuhan Polytechnic University in China. The conference was co-sponsored by ASCE Construction Institute (CI), Transportation Research Board (TRB), and Frontiers of Structural and Civil Engineering in China. The Conference was chaired by Professor Jenny Liu and co-chaired by Professor Hehua Zhu.

We would like to acknowledge the assistance from Donna Dickert of ASCE, and Laura Ciampa and Paul Sgambati of ASCE CI that makes it possible for this high quality peer reviewed STP. The editors are highly indebted to the following individuals who reviewed one or more papers submitted for consideration of publication in this STP:

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Contents

Green Stormwater Infrastructure Strategies for Airports: Challenges and Opportunities

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Abstract

Airports urgently need resilient and affordable solutions to address stormwater quantity and quality issues and to promote the triple bottom line of sustainability. Recent years have seen increasing use of green stormwater infrastructure (GSI) strategies at airports. GSI solutions (e.g., bioretention systems, rain gardens, vegetated filter strips, permeable asphalt or concrete pavement, drainage wells, and amended topsoil) are designed to supplement or replace conventional grey infrastructure (e.g., impermeable pavements and curbs, inlets and pipes) that inhibit water filtration or infiltration and related natural treatment and flow attenuation processes. This work aims to provide a brief overview of the GSI strategies for airports, followed by a discussion of challenges and opportunities in balancing airport priorities in environmental, economic, and social values and operational constraints. The airport challenges in implementing GSI strategies mainly include those related to wildlife attraction, climate change, anti-icing/deicing compounds, and land use limitations. This work presents a synthesis of information that can be valuable in assisting airport decision-makers and professionals responsible for managing the stormwater programs and for the planning and project development of conventional grey infrastructure and new green infrastructure related to stormwater management.

INTRODUCTION

Stormwater regulation has evolved over the past 40 years since passage of the Clean Water Act in 1972. Early legislation focused primarily on control of permitting point discharges to navigable waters through the National Pollution Discharge

Elimination System (NPDES). The limited scope of this initial rule addressed only those major manufacturing facilities with discharges that included potential for contaminated stormwater. Variations and amendments to the Clean Water Act continued until U.S. Congress and the Environmental Protection Agency (EPA) both realized the importance of addressing all stormwater discharges. The Water Quality Act of 1987 provided the framework to establish conceptual classifications of stormwater as either municipal or industrial, which allowed for more focused permitting. Today, there are three main categories of regulated stormwater discharges: municipal, industrial, and construction. Many of the day-to-day operations of airports fall under the purview of several NPDES permits. For example, transportation facilities, identified under the Standard Industrial Classification (SIC) code 45, often conduct vehicle maintenance, equipment cleaning, or aircraft deicing operations. These activities meet requirements for one or more categories defined under industrial discharges. Areas of airports that might not be subject to the industrial program may be regulated by the municipal program such as parking lots, access roads, and commercial operations. Other activities might require permits addressing construction discharge. Navigation of the rules and requirements required by airports when addressing stormwater discharge can be difficult. Although water quality across the country has improved appreciably since 1972, challenges still remain. In 2000, water-quality assessments by states indicated that 39 percent of assessed stream miles, 45 percent of assessed acres of lakes, and 51 percent of assessed estuary areas failed to meet criteria for one or more designated uses (USEPA, 2002). The top causes of impairment in assessed stream miles were siltation, nutrients, bacteria, metals (primarily mercury), and oxygen-depleting substances. Pollution from urban and agricultural land that is transported by precipitation and runoff was found to be the leading source of impairment (USEPA, 2002; Selbig et al., 2013).

Airports urgently need resilient and affordable solutions to address stormwater quantity and quality issues and to promote the triple bottom line of sustainability (as shown in Figure 1). Airports are permitted under the Industrial Permitting and must prepare a Stormwater Pollution Prevention Plan (SWPPP) that requires regular monitoring, site inspections and reporting. Additionally, airport drainage design as directed through the FAA Advisory Circular AC No: 150/5320-5D aims to safely and efficiently remove water from airport premises, to aid in safe travel on runways and other surfaces, and to discourage waterfowl and other wildlife. The quick and efficient removal of potentially polluted stormwater from airport facilities conflicts with the USEPA intent to eliminate pollutants from waterways, unless stormwater treatment and management practices are utilized. The ultimate goal is to enhance the environmental sensitivity of the built environment and improve the relationship between the airport infrastructure and its surrounding environment and local community. While green infrastructure presents great opportunities for airports in their stormwater management efforts, the implementation of green infrastructure must consider constraints related to safety and operations (e.g., standing water or risk of wildlife attraction, accessibility issues, restrictions on some facilities in specified runway zones).

Figure 1. The triple bottom line model (los angeles world airports, 2010).

There has been increasing usage of green stormwater infrastructure (GSI) strategies with a growing body of airport user experience over the last decade or so, but the related information has not been synthesized to facilitate the adoption of GSI strategies into airport design and stormwater management. Experience and knowledge regarding the use of GSI strategies may vary from airport to airport, often related to: the airport size, local land use and vegetation characteristics, precipitation intensity and frequency, drainage area and hydraulic conductivity of surrounding soil layers, typical runoff hydrograph and pollutant profile, operational constraints, and local or regional stormwater regulatory requirements. In this context, this work aims to provide a brief overview of the GSI strategies for airports, followed by a discussion of challenges and opportunities in balancing airport priorities in environmental, economic, and social values and operational constraints.

GSI STRATEGIES: OVERVIEW

The concept of green infrastructure was initially developed by design, conservation and planning disciplines in the 1990s and related to the importance of the natural environment in informing decisions about land use planning. These ideas were captured at the federal level when the President's Council on Sustainable Development identified "green infrastructure" as one of several key strategies for achieving sustainability in the 1999 report entitled *Towards a Sustainable America – Advancing Prosperity, Opportunity and a Healthy Environment for the 21st Century*. More recently, the term has been adopted by the USEPA in relation to the management of stormwater runoff in urban environments. At its core, green stormwater infrastructure strives to use a network of decentralized strategies that mimic natural processes to lower the volume and improve the water quality of stormwater runoff while also creating a healthier urban environment (Sansalone et al., 2013). GSI strategies feature the reduction or transformation of paved surface and the use of "plants, soils, and microbes to collect, treat and reduce stormwater runoff" (Cook, 2007).

Green infrastructure described by the USEPA includes downspout disconnections, rainwater harvesting, rain gardens, planter boxes, bioswales, permeable pavements, green alleys and streets, green parking, green roofs, urban tree canopy, and land conservation (USEPA, 2015). Generally acknowledged classes of benefits of green infrastructure include: water quality and quantity, air quality, energy, climate change, community livability, habitat and wildlife, and public education. Numerous examples (as shown in Figure 2) exist in which creative application of green infrastructure can lead to intriguing benefits. Permeable asphalt or concrete pavements control the quantity and velocity of stormwater runoff, allow for stormwater infiltration, reduce heat-island effect and traffic noise, and recharge the groundwater (Field et al., 1982; English et al., 2008; Xie et al., 2015). Green roofs reduce the volume of stormwater runoff while decreasing building energy usage and reducing noise transmission (Thomas and Wible, 2010). Bioretention practices "remove stormwater pollutants through physical and biological processes including absorption, plant uptake, filtration, microbial activity, decomposition, and sedimentation" (Jones and Jha, 2009) and can be effective in removal of heavy metals (Davis et al., 2003). Bioretention practices also reduce and delay peak flows, recharge ground water basins, and enhance the aesthetics and biodiversity of urban environments. Rain gardens reduce the stormwater volume by infiltration and evapo-transpiration while reducing the pollutant load in stormwater runoff (Flynn and Traver, 2013).

Figure 2. Bioswales, permeable pavement, and rain garden (courtesy of USGS).

GSI STRATEGIES FOR AIRPORTS: EMERGING CHALLENGES

Stormwater problems from airport property can be categorized into (1) impacts from existing impervious surfaces and (2) impacts from newly constructed impervious surfaces. Airport stormwater may induce increased runoff volumes and changes to peak flow rates and timing, posing a risk for downstream receiving waters (e.g., severe erosion and degradation of river and stream channel banks and riparian areas). Typical pollutants may include anti-icing and deicing chemicals used on airfield pavement (e.g., potassium acetate, sodium acetate, potassium formate, corrosion inhibitors and other additives) and on aircraft (typically ethylene glycol or propylene glycol-based products with corrosion inhibitors, thickeners, etc.) (Shi, 2008; Swizenbaum et al., 2001; Astebol et al., 2004; Smith Reynolds Airport, 2010). They may also include petroleum-related compounds, sediment (indicated by total suspended solids), metals (Cu, Zn, mercury, etc.), and nutrients (indicated by biochemical oxygen demand, total P, total N, etc.) (Ingvertsen et al., 2011), as a result of aircraft and other vehicle refueling, vehicle maintenance, material storage and stockpiles, and jet fuel exhaust deposits. The anti-icing and deicing compounds used at airports are highly biodegradable and can exert a high oxygen demand when mixed with stormwater and discharged to water bodies (Fay and Shi, 2012). Low-oxygen conditions in the water body can impair aquatic biota and may result in fish kills (McIntyre et al., 2015). An additional environmental concern with deicer stormwater runoff is toxicity to aquatic biota, mainly associated with product additives and, when urea is used for pavement anti-icing, ammonia. The biochemical oxygen demand (BOD) of typical deicer stormwater runoff ranges from 500-10,000 mg/L. Typical domestic sewage has a BOD of 200-300 mg/L and BOD effluent requirement for publicly owned treatment works (POTWs) are 30 mg/L. Based on recent USEPA technology-based effluent limitation guidelines (USEPA, 2012), new airports with at least 1,000 annual non-propeller aircraft departures must collect 60% of aircraft deicing stormwater runoff. Effluent guidelines for collected stormwater include a maximum chemical oxygen demand (COD) of 271 mg/L, a weekly average COD of 154 mg/L, and if using urea-based pavement anti-icing compounds, ammonia of 14.7 mg-N/L. Discharge requirements at existing airports continue to be established on a site-specific, best professional judgment basis. More detailed information can be found in Airport Cooperative Research Program (ACRP) Report 14 (Deicing Planning Guidelines and Practices for Stormwater Management Systems) and ACRP Report 99 (Guidance for Treatment of Airport Stormwater Containing Deicers).

Increasing regulatory attention is being paid to stormwater runoff from airports, which may include court orders, legal actions and Total Maximum Daily Loads (TMDL) plans. Airports must observe growing local, state, and federal regulations to protect the environment by controlling the quality and quantity of stormwater runoff from airport facilities. But airports are different from typical urban sites since they must pay attention to aircraft safety. One of the main concerns is that many conventional stormwater strategies attract *wildlife*, and this can increase the potential for aircraft bird strikes (Fox et al., 2013). This issue has been recently studied and details are available in ACRP Report 125 entitled *Balancing Airport Stormwater and Bird Hazard Management.* Most aircraft bird strikes take place near airports and many of the bird species involved in strikes, such as waterfowl, gulls and raptors, are attracted to open water. According to the Federal Aviation Administration (FAA), damage related to aircraft bird strikes cost US civil aviation more than half-a-million dollars per year and result in over half-a-million hours per year in aircraft downtime. To minimize the attraction of wildlife, stormwater facilities at airports are commonly designed to limit open water. Even when ponding water is kept at short durations, the vegetation at these facilities may still attract wildlife. Vegetation that is in or around wetlands, detention basins or bioswales should be specific native vegetation so that it does not attract wildlife as a food source or create habitat.

Subsurface wetlands are favored over less complex surface-flow wetlands and infiltration systems are designed to minimize the duration of ponding.

Another issue that is closely tied to GSI strategies at airports is *climate change*. Likely hydrologic changes will include changes in precipitation amounts and increases in precipitation intensity. As acknowledged in the recent report *Adapting Transportation to the Impacts of Climate Change* (Transportation Research Board, 2011), "climate change is likely to have subtle long-term as well as dramatic short-term effects on airports in the future." Airports are uniquely vulnerable to these changes for a number of reasons. Many airports cover a large surface area on the landscape that can encompass multiple watersheds with diverse landscapes, potentially compounding the impacts of changing hydrologic patterns. In addition, stormwater infrastructure investments are often expected to last for decades, but existing systems or new systems that lack resilience could become overloaded as storm events grow in frequency and intensity. Lastly, airports are commonly combined governmental-private enterprises, and this leads to a focus on near-term return on investments as opposed to managing and adapting to changing constraints on the temporal scale of decades. Obviously, GSI strategies will need to be developed and assessed in the context of changing patterns of climate and precipitation.

Furthermore, airports face a number of unique challenges related to the application of natural treatment systems to manage stormwater rich in *anti-icing/de-icing compounds*. One challenge is cold weather, which tends to depress biological processes and associated pollutant removal in natural treatment systems. Another challenge is minimizing open water associated with treatment wetlands to reduce the threat of aircraft bird strikes. Over the past 15 years, a number of natural treatment systems have been tested or installed at airports. Examples include a vertical subsurface flow treatment wetland installed at Toronto's Pearson International Airport, a horizontal subsurface flow treatment wetland installed at Edmonton International Airport (Higgins and McLean 2002), a horizontal subsurface flow treatment wetland installed at Westover Air Reserve Base in Massachusetts, and a hybrid surface-subsurface treatment wetland at London's Heathrow Airport (Revitt et al., 2002). Yet another challenge is maintaining the durability and serviceability of impermeable and permeable asphalt and concrete pavements in the presence of deicers (Pan et al., 2008; Hassan et al., 2002; Goh et al., 2011; Shi, 2008; Sun et al., 2002; Shi et al., 2010 & 2011; Xie et al., 2014).

Finally, stormwater management facilities need to incorporate *land use limitations* within FAA defined areas such as clearways, object-free areas, runway protection zones, runway safety area, and stopways. All FAR Part 139 Airports (e.g. Seattle-Tacoma Airport) are required to have their safety areas compacted to support emergency vehicles, snow removal equipment and the occasional aircraft. This compaction requirement for soils near runways poses an interesting constraint on stormwater management (personal communications, Aaron Moldver, Environmental Management Specialist, 2015). These highly compacted soils are not good at infiltrating stormwater, which adds to the challenge of designing and implementing bioswales. Stormwater facilities in the near-runway environment must be able to support large equipment and may have height restrictions.

There are differences in challenges of implementing GSI strategies at non-primary and primary airports. One difference is related to the regional weather patterns. Plant life needs to be drought-tolerant in some regions and able to withstand extreme cold in other regions. Primary airports in those colder regions use more chemicals to anti-ice/deice runways and aircraft than those non-primary airports. The non-primary airports may only use mechanical means to battle icing conditions on airfield pavement and may not even need to remain open during severe snow storm events. Larger airports may have more non-pervious surfaces, which increases the amount of stormwater requiring treatment. Depending on the aircraft operations, primary airports serving turbine-powered aircraft have greater a separation distance requirement from hazardous wildlife attractants than those of the smaller airports serving piston engines (FAA Advisory Circular 150/5200-33B Hazardous Wildlife Attractants on or near Airports), and therefore, have much greater challenges in implementing GSI in the operational areas. Unlike the larger airports, smaller airports have difficulties minimizing wildlife hazards on and around airports due to lack of dedicated staffing and funding. Larger primary airports have funding opportunities to implement GSI strategies located in the landside part of the airport using green roof rainwater harvesting, bioswales and permeable pavements in parking lots.

GSI STRATEGIES: OPPORTUNITIES

Environmental managers are often faced with the task of designing strategies to accommodate expanding development while minimizing adverse environmental impacts. Low impact development (LID) is one such strategy that attempts to mitigate environmental degradation commonly associated with traditional residential or commercial construction practices. LID, or what is often referred to as green infrastructure, attempts to maintain or replicate the predevelopment hydrologic regime through use of design techniques that create a functionally equivalent hydrologic landscape. LID principles are based on controlling stormwater at the source by incorporating the hydrologic functions of storage and infiltration into small-scale, distributed, structural and nonstructural controls. Some LID practices include, but are not limited to, decreasing impervious surfaces through narrower streets, permeable pavements, creating micro-scale stormwater retention and detention areas such as rain gardens or bioretention cells, increasing flow paths, preserving highly permeable soils, incorporating vegetated swales and permeable paving into the building plan, and preventing soil compaction by discouraging the use of heavy equipment (Coffman, 2000; Liaw et al., 2000).

Airports are increasingly exploring the use of green infrastructure strategies and tactics for addressing their stormwater challenges, as the appropriate integration of green infrastructure into new construction or planned improvement projects can result in significant cost savings and environmental benefits (Faucette, 2002; Nickel et al., 2014). Green infrastructure solutions (e.g., bioretention systems, rain gardens, vegetated filter strips, permeable asphalt or concrete pavement, drainage wells, and amended topsoil) are designed to supplement or replace conventional grey infrastructure (e.g., impermeable pavements and curbs, inlets and pipes) that inhibit water filtration or infiltration and related natural treatment and flow attenuation processes. Green stormwater infrastructure strategies aim to restore, protect, and mimic natural hydrologic functions within the built environment. Examples include a green roof and permeable asphalt parking lot installed on the Heritage Aviation facility located at the Burlington International Airport, Vermont.

Figure 3. Bioinfiltration (*left*) and pervious pavement (*right*).

Local municipalities have increasingly implemented these innovative solutions and demonstrated their potential at greatly reducing the water quantity and quality impacts of stormwater (Keeley et al., 2013). Federal and state regulatory agencies are also promoting green infrastructure strategies as a promising approach to complying with their water regulations and requirements. Green infrastructure techniques, technologies, and elements follow the principles of context-sensitive solutions (CSS) and LID in design and project development (English et al., 2008; Zhou et al., 2013; Cettner et al., 2014), and they work by weaving the natural or simulated natural processes (e.g., infiltration) into the built environment, as shown in Figure 3. Other themes relevant to GSI strategies include stormwater best management practices (BMPs) and life cycle analysis (LCA). "Green" projects focus on the environmental stewardship component of the Triple Bottom Line (Figure 1) with the multiple goals of minimizing pollution and waste and limiting water, energy and carbon footprints. Stormwater BMPs such as ponds, wetlands, and vegetated swales and filter strips, can still remove high levels of sediment from pavement runoff if designed, sited, installed, and maintained properly (Staples et al., 2004; Mericas et al., 2009; Jungwirth et al., 2015). LCA includes a whole-life impact assessment of projects including construction and operations and maintenance (O&M) costs and impacts.

One GSI strategy with potential for managing deicer stormwater runoff is natural treatment systems – surface-flow or subsurface-flow vegetated systems which can act both as treatment systems and flow equalization basins. A number of wetland treatment systems have been installed at airports to manage stormwater runoff, but an acknowledged limitation of these systems are their low oxygen-transfer capacity, an especially critical factor when treating stormwater runoff with high oxygen demand. *Is there a way to integrate the simplicity of the natural treatment system with the effectiveness of the highly engineered aerated gravel bed?* This can be achieved through the novel combination of oxygenation technology with subsurface natural treatment systems with passive nutrient addition. Coupling of oxygenation technology with treatment systems for deicing stormwater runoff is an acknowledged research gap detailed in ACRP Report 99. As noted earlier, a key limitation of BOD removal in subsurface wetlands is oxygen availability. Typical subsurface wetlands can supply around 20 $g/m^2/d$ of oxygen while aerated subsurface systems can deliver up to 180 $g/m^2/d$. However, aerated subsurface systems have complex piping systems susceptible to clogging. By using a side-stream oxygenation approach, a more simple and inexpensive subsurface treatment wetland system can be used. In addition, oxygenation overcomes the conundrum of low oxygen transfer efficient associated with aeration. Since the driving force for oxygen transfer when using air is the difference between the saturation concentration (~10 mg/L) and actual water concentration, oxygen transfer efficiency decreases and energy use increases markedly as dissolved oxygen (DO) concentration in water approaches saturation. By using pure oxygen gas, this is no longer an issue since DO saturation can be > 50 mg/L. As a result, oxygen transfer efficiencies are high and associated energy cost to dissolve DO into water are low.

To overcome limited removal kinetics in cold weather and high oxygen demand in stormwater with deicers, some researchers have proposed the use aerated wetlands, which were recently tested at Buffalo-Niagara International Airport (Higgins et al., 2011). Pure oxygen gas has been proposed as an effective source of oxygen for treatment wetlands (Palmer et al. 2009; Allen et al. 2010) and there are a number of full-scale oxygenated wetlands treating industrial wastewater (David Austin, CH2M Hill, personal correspondence, 2015).

CONCLUDING REMARKS

This work has presented a state-of-the-knowledge review on the use of green stormwater infrastructure strategies at airports, including a general overview, emerging challenges, and opportunities. Multiple themes (CSS, LID, stormwater BMPs, and LCA) are related to GSI strategies and they could be integrated into a holistic tool to assess and implement “green” stormwater infrastructure at airport facilities. The airport challenges in implementing GSI strategies mainly include those related to wildlife attraction, climate change, anti-icing/deicing compounds, and land use limitations. It may be necessary to implement policy changes to incentivize the incorporation of green infrastructure into airport design or stormwater management program. Public private partnerships can be instrumental in moving things forward.

Of great interest is the use of GSI strategies that are not an attractive nuisance, as to comply with FAA Advisory Circular AC No: 150/5200-33B *Hazardous Wildlife Attractants on or near Airports*. One interesting GSI strategy discussed is oxygenated subsurface wetlands for the treatment of deicer stormwater runoff from airports. Alternatively, it may be necessary to develop plant species lists that included less palatable species such as an endophyte inoculated fescue.

Managing stormwater with a distributed green infrastructure network requires assessing the airport landscape comprehensively. GSI may be implemented as stand-alone system, or hybrid system, or complement to existing grey infrastructure. The selection of appropriate GSI for the given airport depends on many factors, such as: local planning and zoning stormwater issues, typical stormwater runoff pollutants and their loading, local precipitation, vegetation and soil characteristics, and land use limitations. It is also crucial to consider the costs, performance, life expectancy, and operational constraints of GSI strategies, relative to conventional grey infrastructure strategies. Some GSI strategies have not yet been implemented by airports for sufficient time and the lack of long-term maintenance and effectiveness data makes it difficult to assess their life-cycle cost or environmental impact.

It is important to capture the insights of airport planners, managers, engineers, and possibly contractors regarding the advantages, disadvantages, cost-effectiveness, and operational constraints of various GSI strategies. This is part of the scope by the ACRP 02-64 project (http://apps.trb.org/cmsfeed/TRBNetProjectDisplay.asp?ProjectID=3835, last accessed on March 1, 2015) which will develop a GSI primer and guidebook for airports and other stakeholders.

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Determination of Ground Water Resources in Elekuro and Environs, Abeokuta: Using the Geoelectric Method (Vertical Electrical Sounding)

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Abstract

Geophysical investigation of Elekuro and environs, Abeokuta, South western Nigeria was undertaken to explore the potential for further ground water development, considering the depth, thickness, resistivity and layer at which water can be obtained. The geo-electrical method used in the survey is Vertical Electrical Sounding. Six Vertical Electrical Soundings (VES) were conducted using the Schlumberger configuration covering the entire area. The VES data were subjected to an iteration software (WIN RESIST) which showed that the area is composed of top soil, weathered layer, fractured layer and basement. The result of a quantitative interpretation of the VES data obtained in a geophysical survey are represented by a geoelectric section which shows the sequence and relationship between the subsurface lithologies. The weathered layer and the fractured zone have been identified as the aquiferous in the area. The weathered layer is thicker in VES 1 and thinnest in VES 2, while the fracture basement is thickest in VES 1. Based on low resistivity values, overburden thickness and Reflection coefficient, VES1 has the highest and brightest potential for future groundwater exploration and development in addition to the existing ones.The study area falls within the basement complex of Southwestern Nigeria. The rocks are Precambrian in age as they lie between the pan African orogenic belts.

INTRODUCTION

Vertical electrical soundings, when combined with other geophysical methods, geologic mapping and available well data can greatly assist in the location and completion of water wells in bedrock areas of complex hydrogeology. The vertical electrical sounding (VES) method is usually considered more suitable for the subsurface investigation of geologic environments consisting of horizontal or nearly horizontal layers, such as occur in unconsolidated sedimentary sequences. Abrupt lateral changes in lithology and electrical properties brought about by steeply dipping beds, fracture and fault zones, or highly variable thicknesses of weathered bedrock materials can make interpretation of VES admittedly very difficult in this type of geologic setting.

However, with appropriate field techniques in completing the soundings combined with the application of appropriate geoelectrical and geological models, VES results can focus test drilling at locations and to target depths which will result in successful water wells, even at sites where "dry" or marginal wells were previously drilled. The case history presented herein illustrates one typical successful application.

In Nigeria, Crystalline rocks cover about fifty percent (50%) of the total rock exposure and about seventy percent (70%) of these crystalline rocks belongs to the basement complex (Oyawoye, 1964). The basement complex rocks of Nigeria are well represented on the south western part of the country. Elekuro and it environs a town in Abeokuta area south western Nigeria; it is underlain by some of the crystalline rocks described by Oyawoye (1964). These rocks belong to the youngest of the three major province of the West African craton; these rocks were rejuvenated during the pan African thermo-tectonic orogeny about 600ma Dada, (1993). Some of the outcrops are quite resistant to weathering while few of them show differential origin on surface. The various rocks units represented in the study area correspond to the generalized list of the basement complex rocks of Nigeria. The occurrence of the ground water in these areas is depended in the degree of fracturing and/or weathering of the basement complex rocks also the chemistry (quality) of the ground water is intimately to the geology and anthropogenic factors.

In this paper geophysical investigation was undertaken to explore the potential for further ground water development around Elekuro and environs, Abeokuta, South western Nigeria. Electrical resistivity vertical sounding is utilized. Field mapping was done and subsequent production and the geological map of the area was gotten. This exercise was done and directed towards carrying out geophysical investigation of groundwater in the study area. It is also aimed to identify different rock type and determine the mineral composition of the rocks, delineating the several geologic units beneath the surface of the area, ascertaining their hydro-geologic significance, determine the suitability of drilling a productive borehole at the points investigated and make appropriate recommendation to ensure the success of the borehole.

The vertical electrical sounding method was selected for this study because of its capability to distinguish between saturated and unsaturated layers, it has a greater penetration than the Wenner. In the resistivity method the Wenner discriminate resistivity of different geoelectric lateral layers while schlumberger configuration is used for depth sounding (Olowofela et al. 2005)

Geological setting of the study area. The study is located between longitude 3^0 18'-3^0 21' and latitude $7^0$07'-$7^0$09'. It is a portion of south/Eastern part of Abeokuta South/Western Nigeria**.** The area fall within the basement complex. It covers a total of 21km^2 area. The study area include Aro, Apena, sogeke and Molekero as shown in Figure 1. The topography of the study area is generally undulating ranging from high to low relief. The drainage pattern exhibit by the stream and rivers can be described as being dendritic, which is the main river that divides the study area into half but most of the streams were dried up because of the dry season which made the area easily accessible.

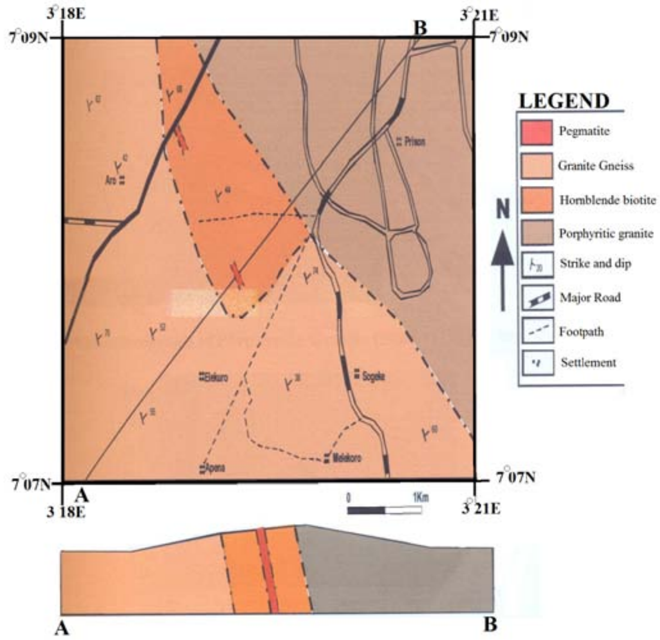

Figure 1. Geologic map showing the study area.

METHODOLOGY

Survey method and electrode configuration. Vertical Electrical Sounding was carried out using the Syscal R2 unit terrameter for the survey. The Schlumberger electrode configuration was selected for this study due to its logistic man power, ability to distinguish between saturated and unsaturated layers and its characteristic deep penetration into the subsurface.

The Schlumberger array is characterized with a more complex configuration with the spacing between the current electrodes not equal to the spacing between the potential electrodes. The schlumberger array requires that four steel electrodes are arranged and pinned collinearly into the earth with the current electrode spacing much greater than the potential electrode and ensuring that $AB/2 \geq 5MN/2$, where "AB" is current electrodes separation and "MN" is the potential electrode separation (Figure 2).

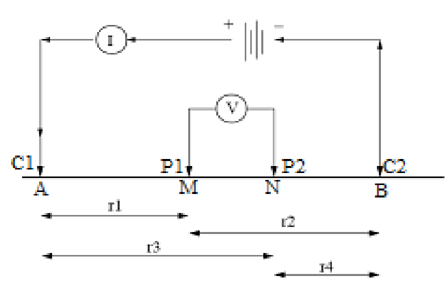

Figure 2. Schlumberger configuration ($AB/2 \geq 5MN/2$).

Procedure. Six points were investigated in the area as shown in Figure 3. These points were selected based on the grids on the map for Vertical Electrical Sounding (VES) using the Schlumberger Array.

The Schlumberger array was used for the VES operation. This array is capable of isolating successive hydrogeologic layers beneath the surface using their resistivity contrast. The method has been recognized to be more suitable for hydro geological survey of sedimentary basin (Kelly and Stanislav 1993).

A maximum of 100 m was attained for the current electrode spacing ($AB/2$) at the two locations. In VES, with schlumberger, the potential electrodes are moved only occasionally, and current electrode are systematically moved outwards in steps $AB \geq 5MN$. The field data obtained are resistance values for the points investigated; the apparent resistivity was calculated for all the sample points using their corresponding geometric factors.

The values obtained were then plotted on a log-log paper as points with the resistivity values being on the vertical axis and the electrode spacing ($AB/2$) on the horizontal axis. The points were joined and curve matched manually using pre-calculated master curves and their auxiliaries. The results obtained from the exercise were used as the input-model for the eventual computer aided iteration (Vanda Velpen, 1988) WINRESIST program.

RESULTS AND DISCUSSIONS

The locations of the soundings are shown on Figure 3, the sounding curves and interpreted geoelectric models are shown in Figure 4a-4f and 6 respectively. The soundings were made using a Syscal R2 unit Schlumberger Array. Current electrode spacing (AB/2) varied from 1-100m

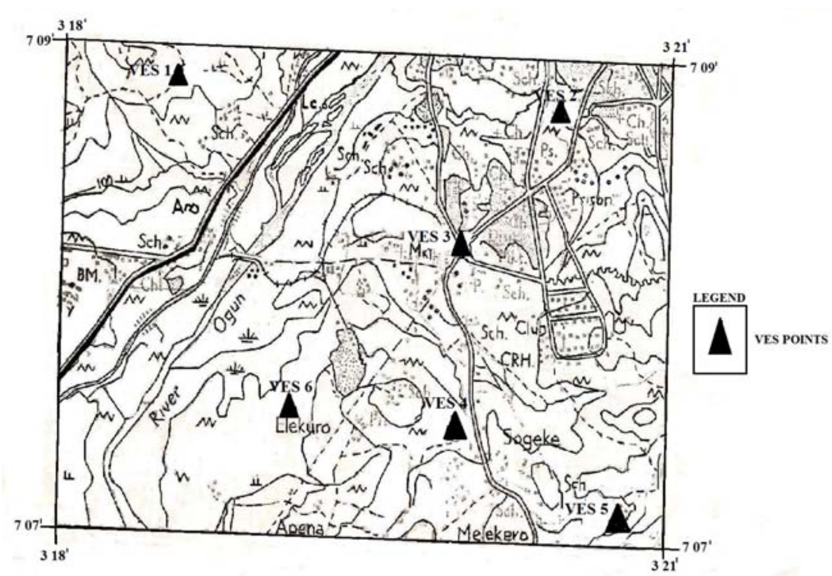

Figure 3. Showing the VES points and Water samples location.

Results and interpretation. The field data obtained from the surveyed locations is presented in Table 1, The interpretation of the sounding curves were done both qualitatively and quantitatively. The qualitative interpretation entails the observation of the sounding curves as plotted on the bi-logarithm graph paper.

Data interpretation. An iteration software (WIN RESIST) is used to iterate curves of VES 1-6 and it is represented in Figure 4a-4f. The smooth curves taken through the set of data points were interpreted quantitatively by the method of partial curve matching. Layer resistivity and thickness were gotten from Figure 4a-4f.

Table 1. Resistivity values with their corresponding electrode spacing.

SN	AB/2(m)	VES 1	VES 2	VES 3	VES 4	VES 5	VES 6
1	1	354.8	100	526.3	170.6	120.8	121.8
2	2	126.8	182	392.4	162.9	97.1	60
3	3	74	212.2	340	173.6	78.9	46.4
4	4	50.2	206.4	300	180.4	80.6	46.4
5	5	45	166.7	200	158.6	100	48
6	6	62.2	135	130	135.5	124.9	61.8
7	12	100	95	120	117	143.3	75
15	15	115.1	78.8	120	100	150.5	90
8	20	130	78.6	140	101	141.1	118.7
9	25	135	85	160	120	144.1	144.7
10	32	139.6	100.7	190	86.7	136	170
11	40	160	136.6	240	248	130	230
12	65	195.6	158.6	253.7	298.9	120.7	286.5
13	80	210	210	290	374.1	119.5	356.1
14	100	309	250	335.3	467	125	423.9

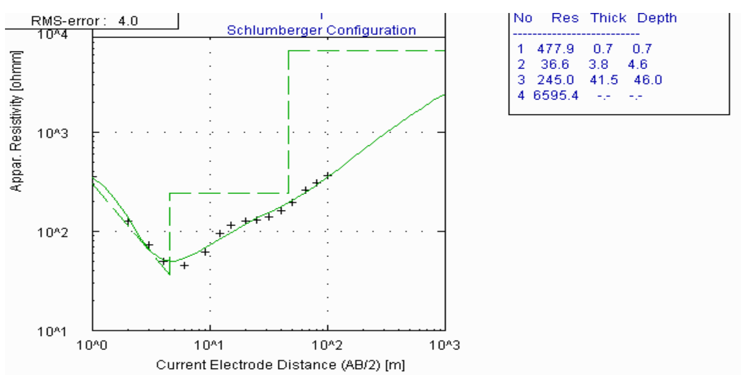

Figure 4a. Field sounding curve for point VES 1 in Elekuro and environs.

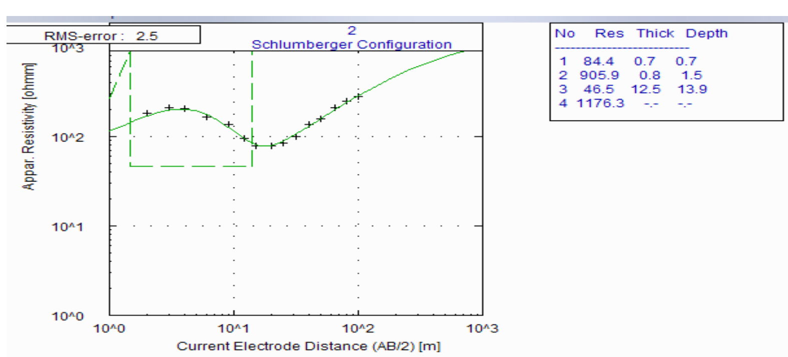

Figure 4b. Field sounding curve for VES 2 in Elekuro and environs.

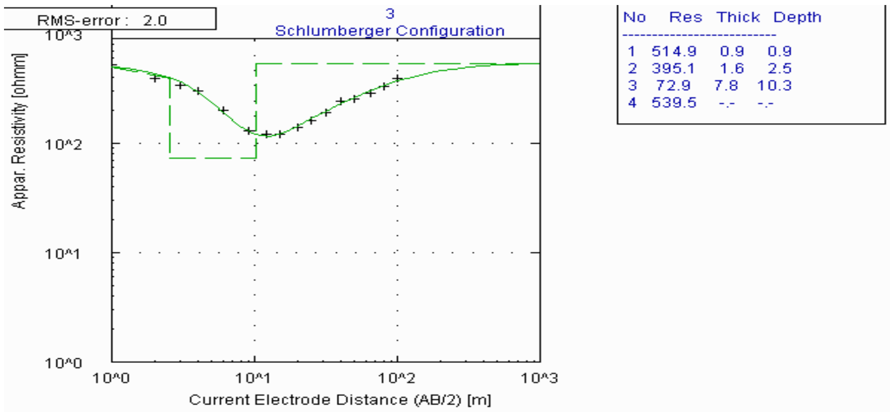

Figure 4c. Field sounding curve for point VES 3 in Elekuro and environs.

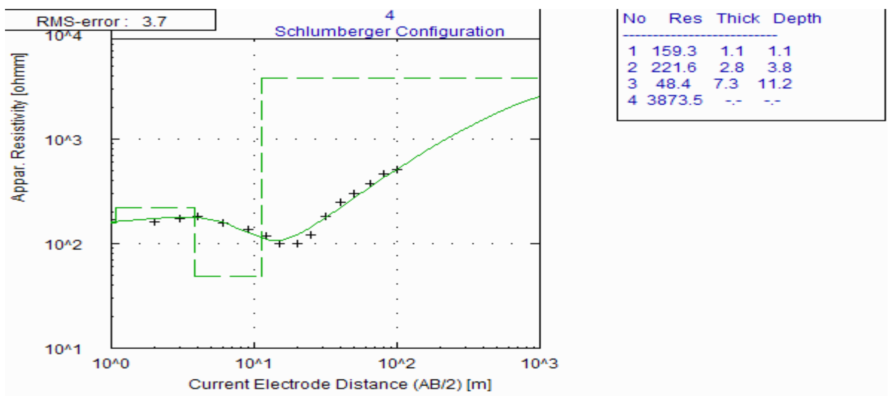

Figure 4d. Field sounding curve for point VES 4 in Elekuro and environs.

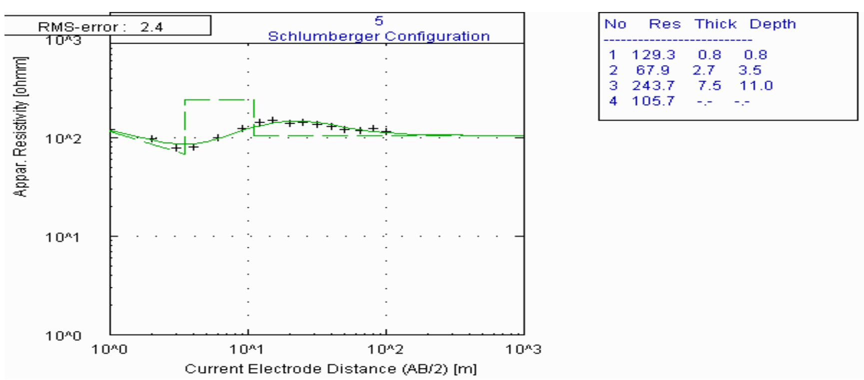

Figure 4e. Field sounding curve for point VES 5 in Elekuro and environs.

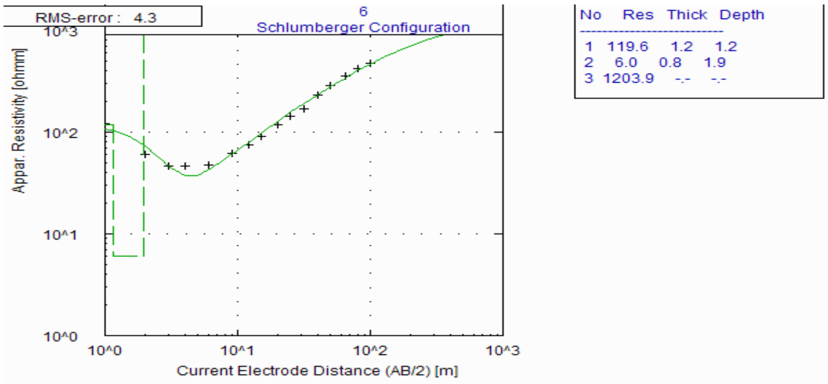

Figure 4f. Field sounding curve for point VES 6 in Elekuro and environs.

Table 2. Result of vertical electrical sounding showing the layers, thickness, depth, resistivity and inferred lithology.

VES	Layer	Resistivity Ωm	Thickness (m)	Depth (m)	Inferred Lithology
1	1	477.9	0.7	0.7	Top soil
	2	36.6	3.8	4.6	Weathered layer
	3	245.0	41	46.0	Fractured layer
	4	6595.4	-.-	-.-	Fresh basement
2	1	84.4	0.7	0.7	Top soil
	2	905.9	0.8	1.5	Weathered layer
	3	46.5	12.5	13.9	Fractured layer
	4	1176.3	-.-		Fresh basement
3	1	514.9	0.9	0.9	Top soil
	2	395.1	1.6	2.5	Weathered layer
	3	72.9	7.8	10.3	Fractured layer
	4	539.5	-.-	-.-	Fresh basement
4	1	159.3	1.1	1.1	Top soil
	2	221.6	2.8	3.8	Weathered layer
	3	48.4	7.3	11.2	Fractured layer
	4	3873.5	-.-	-.-	Basement
5	1	129.3	0.8	0.8	Top soil
	2	67.9	2.7	3.5	Weathered layer
	3	243.7	7.5	11	Fractured layer
	4	105.7	-.-	-.-	Fresh basement
6	1	119.6	1.2	1.2	Top soil
	2	6.0	0.8	1.9	Weathered layer
	3	1203.9	-.-	-.-	Fresh basement

Reflection co-efficient. The reflection coefficients (r) of the study area were calculated using the method of Olayinka (1996), Bhattacharya and Patra, (1968) and Loke, (1999). Increase in the Reflection co-efficient and overburden thickness enhances the productivity of boreholes in some parts of the basement complex of southwestern Nigeria (Olorunfemi and Oloruniwo, 1985). Reflection co-efficient can be calculated using equation 1.The reflection coefficient, curve type and characteristics and their respective number of layers are shown in Table 3.

$$\text{Reflection Coefficient (r)} = \frac{R_n - R_{n-1}}{R_n + R_{n-1}} \quad (1)$$

where R is the resistivity value and "n" is the amount of layers. Based on low resistivity values, overburden thickness and Reflection coefficient, VES1 has the highest and brightest potential for future groundwater exploration and development in addition to the existing ones.

Table 3. A table showing the co-ordinates and reflection co-efficient with their corresponding curve types.

Reflection Coefficient	Name/Location	Curve Type	Curve Characteristics	No of layer
0.93	VES 1	H	$\rho_1>\rho_2<\rho_3$	3
0.92	VES 2	KH	$\rho_1<\rho_2>\rho_3>\rho_4$	4
0.76	VES 3	H	$\rho_1>\rho_2<\rho_3$	3
0.97	VES 4	KH	$\rho_1<\rho_2>\rho_3>\rho_4$	4
-0.39	VES 5	HA	$\rho_1>\rho_2<\rho_3<\rho_4$	4
0.99	VES 6	H	$\rho_1>\rho_2<\rho_3$	3

The interpretation of field resistivity data are in terms of resistivities and depth to the bedrock and interfaces across which a strong electrical exists. The analysis and interpretation of the surveyed data shows 3-4 geoelectric layers as shown in Table 2. These layers are lateritic top soil which consists of the various rock types from clayey sand to sandy clay to compact sand. The second is the fresh/highly resistive basement. The fresh basement is characterized by high and infinite resistivity value and could not be contended on for groundwater, but the weathered zone and fractured basement which have lower resistivity value constitute good water zone.

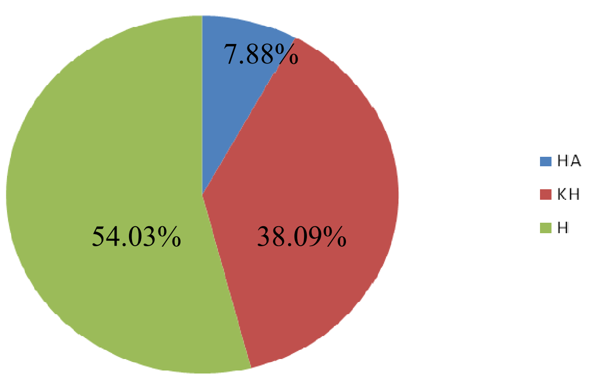

Figure 5. Pie Chart Presentation of Resistivity Curve Types.

The pie chart shown in Figure 5 is a statistical representation of the resistivity curves. The H type is dominant accounting for about 54.03%, with KH and HA type having 38.09% and 7.88% respectively, The H curve has an intermediate layer of low resistivity value that is recognized as the aquifer unit at these VES 1,3 and 6 location. The first layer is the topsoil, followed by a sandy formation and the weathered layer in that order. The weathered layer in these sequences is very favorable for underground water abstraction.

The KH type curve is a four layer model of the subsurface and was obtained at VES 2 and 4. The HA curves in VES 5 is typified a typical basement complex environment which contains a low resistivity layer underlined and overlain by a more resistive materials. (Olayinka and Mbachi, 1992).

Geoelectric Section. The VES results of the data delineates 3-4 layers (H, HA, and KH) types of curve with the H type of curve predominating. The geo-electric section of the study area was produced, revealing 3 main geo-electrical layers; the top soil, the weathered layer (made up of clayey, sandy clay and clayey sand) units and the infinite basement rock as illustrated in Figure 6.

The quantitative interpretation of the VES data resulted in the production of numbers of geoelectric section. The section provides composite information along lithologic depth, the geoelectric section revealed four subsurface geoelectric layers. The top layer which consists of (clayey sand and sandy clay) has resistivity value ranging from 84 ohm-m to 514.9 ohm-m, the maximum layer thickness is 1.1m..

The resistivity of the second layer (Weathered zone) ranges from 6 ohm-m to 905.9 ohm-m while the thickness varies between 0.7m to 12.5m. The third layer is the fractured basement which has layer thickness varying from 7.3m-41.5m with resistivity value ranging between 46.5 ohms-m to 1203.9 ohms-m, The layer will be good for groundwater accommodation if the fractures are interconnected and permeable. The fresh basement which is the fourth layer is characterized by high resistivity value up to 6595 ohm-m. The fresh basement is made up of infinitely resistivity rock in all the stations which form the bedrock.

Overburden Layer. The thickness of the overburden is an important hydrogeologic consideration in groundwater development in the basement terrain (Ajayi & Hassan 1990, Olorunfemi & Idonigie, 1992). Because water gets into the saturated zone through the overburden, the thickness of the overburden ranges from 1.5m to 4.6m in the study area (Figure 6).

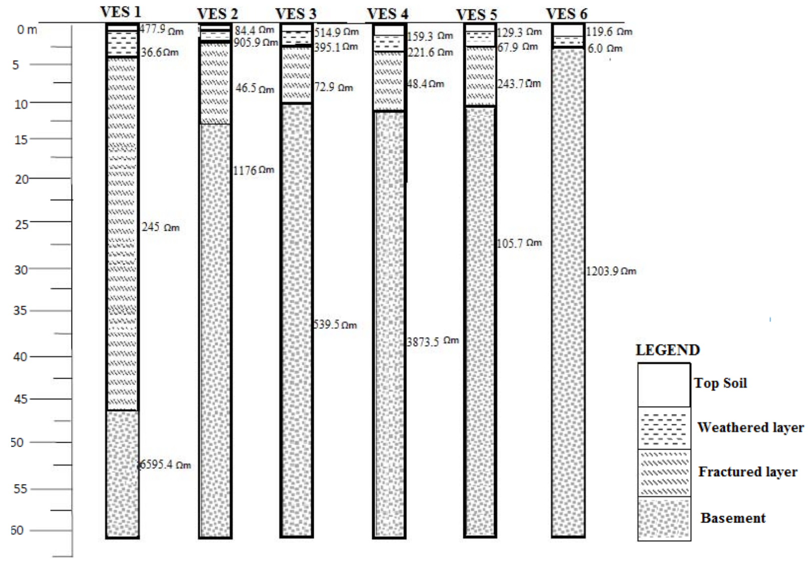

Figure 6. Geo-electric section of profile 2.

The geo-electric section also helped out in relating resistivity with geology by delineating the varying thickness (es) of each layer. It also aids comparison between the different layers from one point to the other in the study area

CONCLUSION

This geophysical investigation of groundwater project benefited from the application of hydrogeology and geophysical surveys, with particular emphasis on the role of VES to characterize favorable subsurface environments for groundwater production. In the complex bedrock geologic environment of the Elekuro and environs, the need for costly random drilling, resulting in dry holes or marginal production from wells can largely be eliminated by the judicious application of these kinds of geological and geophysical studies. The result of a quantitative interpretation of the VES data obtained in a geophysical survey over part of Abeokuta, on a location Elekuro and Environs South Western Nigeria are represented by a geoelectric section which shows the sequence and relationship between the subsurface lithologies. The weathered layer and the fractured zone have been identified as the aquiferous in the area. The weathered layer is thicker in VES 1 and thinnest in VES 2, while the fracture basement is thickest in VES 1. The H curve type is dominant, accounting for about 54.03%, with HA and H curve types having 38.09% and 7.88% respectively, The H curve has an intermediate layer of low resistivity value that is recognized as the aquifer unit at these VES 1 location. The first layer is the topsoil, followed by a weathered layer and The KH type curve is a four layer model of the subsurface and was obtained at VES 2 and 4. The HA curves in VES 5 is typified a typical basement complex environment which contains a low resistivity layer underlined and overlain by a more resistive materials. (Olayinka and Mbachi, 1992). The H curve has an intermediate layer of low resistivity value that is recognized as the aquifer unit at these VES 3 location. The first layer is the topsoil, followed by a weathered and the fractured layer in that order. The weathered layer in these sequences is very favorable for underground water abstraction. Based on low resistivity values, overburden thickness and Reflection coefficient, VES1 has the highest and brightest potential for future groundwater exploration and development in addition to the existing ones. In conclusion, the study area falls within the Basement Complex of Southwestern Nigeria. The rocks are Precambrian in age as they lie between the pan African orogenic belts.

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Quality Control of Precipitation Data for Wet Pavement Accident Analysis

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Abstract

Rainfall and wet pavement have significant impacts on road safety. Wet percent time was found to be an important variable in wet pavement accident analysis. However, the accuracy of wet percent time can be impaired by data with quality issues, which have negative impacts on the identification of high wet collision locations. Hence, data quality control is a necessary and essential part of any weather station network used for generating wet percent time. The absence of quality control can result in poor quality data that severely limits their usefulness. This study showcased the first application of a methodology for the multi-step quality control of precipitation data. The methodology includes data preprocessing, various quality checks, and treatment of missing data. Historical hourly precipitation data reported by rain gauges were obtained from five network data sources to demonstrate this methodology. These five network sources provided data from over 2,000 weather stations in California. The results of quality control were employed by the California Department of Transportation to update its wet percent time information. The methodology can be adopted by other states conducting similar studies or for applications such as climate change research.

INTRODUCTION

Adequate pavement skid resistance is critical to ensure highway safety. The various factors that affect pavement skid resistance include pavement surface condition, traffic volume and speed, locational attributes (i.e., slopes, sharp curves, intersections), and, of interest in this research, pavement wetness (AASHTO, 1976; Harwood et al., 1988).

Statistics indicate that rain and wet pavement have more significant impacts on road safety than snow and ice. In 2001, nearly 79 percent of weather-related crashes in passenger vehicles in the nation occurred on wet pavement and nearly 49 percent happened during rainfall (Wang et al., 2008). During 1976 and 1977, it was found that 13.5 percent of all fatal accidents occurred on wet pavement, while precipitation occurred only about 3.0 to 3.5 percent of the time nationwide (NTSB, 1980). Between 2000 and 2009, 12.6 percent of fatal crashes occurred on wet pavement nationwide (McGovern et al., 2011). In California, there were about 9 percent of the fatal collisions occurred when the pavement conditions were wet in 2003 (Wang et al., 2008). Each fatal crash leads to substantial costs in terms of human suffering and financial loss. According to the estimates by the California Department of Transportation (Caltrans), a one percent reduction in wet pavement collisions in California would save about $13 million, based on 2001 collision statistics (Wang et al., 2008).

To reduce the rate and severity of collisions and minimize property damage, it is critical to identify high wet collision concentration locations in order to investigate and determine probable causes and to recommend safety improvements where they are deemed appropriate and cost-effective. Wet percent time and other factors (i.e., wet pavement exposure) have been used for decades for the identification of wet pavement accidents (e.g., Hankins et al., 1971; Smith and Elliott, 1975; Holbrook, 1976; Runkle and Mahone, 1976; Dierstein, 1977; Blackburn et al., 1978; NTSB, 1980; Kamel and Gartshore, 1982; Dahir and Gramling, 1990; Collins and Pietrzyk, 2001; NYDOT, 2002; VDOT, 2007; FDOT, 2007; KYTC, 2011; FHWA, 2011). Wet percent time refers to the proportion of time during which pavement is damp enough to cause traffic accidents and is measured on an hourly or a daily basis. Karr and Guillory (1972) defined wet time as the total number of hours during which a measurable amount (≥0.01 inches or 0.25mm) of rainfall occurred. This total time is subsequently used to calculate the percentage of time per year during which measurable rainfall events occur. The percentage of rainfall-event time per year (wet percent time) is calculated as:

$$P_{w0.01} = \frac{\text{number of hours with HPD >= 0.01in}}{\text{number of hours in a year}}$$

where HPD = Hourly Precipitation Data.

The value of 0.01 inches (0.25mm) per hour is used as the minimum amount of rainfall necessary to keep a pavement damp for one hour. As such, trace amounts of rainfall were not considered by this research.

Vehicle exposure to wet pavement (wet pavement exposure) is calculated by the product of wet percent time and vehicle miles traveled (Karr and Guillory, 1972; Blackburn et al., 1978; Peters et al., 1980):

$$W_E = (P_{w0.01}) \times (AADT) \times (days) \times (miles)$$

where W_E = Estimated Wet Exposure for the year
$P_{W\,0.01}$ = Percent of hours with 0.01" or more precipitation
$AADT$ = Annual average daily traffic
It reflects the rate of wet pavement accidents in a region (Peters et al., 1980):

$$Wet\text{-}accident\ Rate = A_w \times 10^6 / (L \times ADT_w \times T_w)$$

where

A_w = Number of accidents reported occurring on wet pavement in a particular period

L = Length of section considered in miles

ADT_w = Average daily traffic volume on wet pavement for the same period, obtained by multiplying percent of time pavement is wet from map by AADT

T_w = Number of wet days in period

Another wet pavement factor, Wet Accident Index (WAI) or Wet Safety Factor (WSF), was also used in the literature (Peters et al., 1980; FHWA, 2010) as an appropriate metric for evaluating the effectiveness of a wet weather crash reduction factor:

$$WAI = (A_w \times P_d) / (A_d \times P_w)$$

where

P_d = $100 - P_w$

A_w = Number of accidents on wet pavement

A_d = Number of accidents on dry pavement

P_w = Percent time pavement is wet

P_d = Percent time pavement is dry

Note that $WSF = 1/WAI$. If $WAI = 1$, there is no difference between wet and dry accidents; if $WAI < 1$, the accident risk on wet pavement is less than on dry pavements; and if $WAI > 1$, the accident risk on wet pavement is greater than on dry pavements.

Wet percent time is the basis for developing wet pavement factors (e.g., wet pavement exposure, WAI). The calculation of wet percent time is simple according to the definition, while the accuracy is often impaired due to data quality issues. In order to accurately identify high wet collision locations, it is essential to perform quality control (QC) before estimating wet percent time. In light of this, the purpose of this study is to synthesize the findings from the literature in QC and to propose an appropriate methodology for controlling precipitation (rainfall in this case) data. The methodology is interpreted by using real world data collected from numerous weather stations and various sources.

METHODOLOGY AND CASE STUDY

Based on the causes of hourly precipitation data errors and existing methods for data QC, a procedure to control hourly precipitation data is proposed, as shown in

Figure 1. Overall, the QC includes five components: data preprocessing, Level-I, Level-II, Level-III QC, and treatment of missing data. To better illustrate the QC elements in each component, real world data from various weather data sources were collected for the State of California. The details on how to control data quality are provided along with the analysis of collected data.

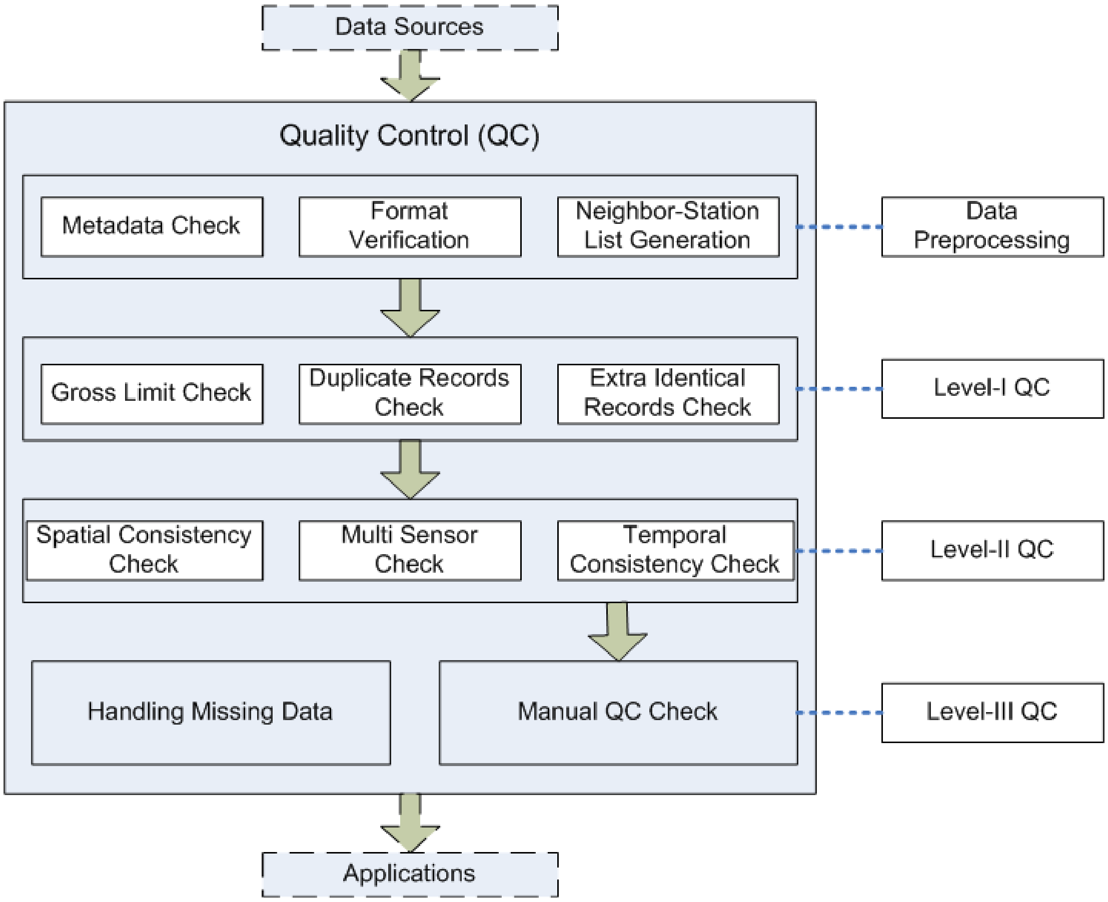

Figure 1. Quality control method.

Data collection. Historical hourly precipitation data of California reported by rain gauges were obtained from five network data sources, including:

- CDEC: California Data Exchange Center (523 weather stations)
- CIMIS: California Irrigation Management Information System (153 stations)
- MESOWEST: University of Utah (1060 stations)
- NCDC: National Climatic Data Center (397 stations)
- NWS: National Weather Service (512 stations)

Historical weather data were acquired from these sources via a number of approaches (computer graphical interfaces, FTP downloads, etc.). Data from CIMIS, NCDC and NWS, were downloaded directly through their web interface. The large amount of data available and acquired from CDEC and MESOWEST were obtained via download through a File Transfer Protocol (FTP) portal.

Based on the density and distribution of available precipitation data stations, an 11-year period was chosen for this study, beginning on January 1, 1995 and extending through December 30, 2005. A total of 2,645 weather stations with hourly precipitation data were available from the previously listed sources. The spatial distribution of these stations is shown in Figure 2. Note that this map was generated using the raw metadata provided by each source and may contain errors in latitude and longitude (as indicated by the points displayed outside of the state's borders).

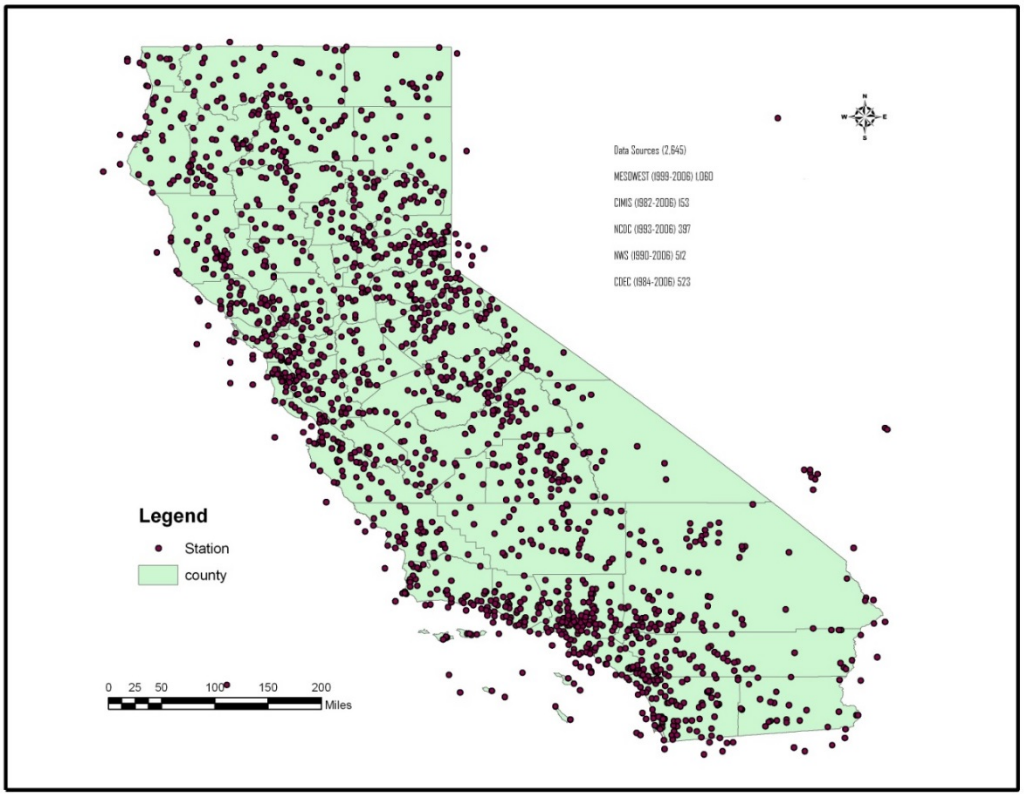

Figure 2. Spatial distributions of stations.

QC of these data sets is an important procedure that must be undertaken before the data can be used for any application. The absence of QC procedures may result in data with poor quality (outliers, missing records) or noises (inconsistent or unrepresentative records) that severely limit their usefulness for applications.

Data preprocessing. Data preprocessing activities include metadata checking, data reformatting, and neighborhood database generation. The data collected from the networks included metadata and precipitation data. Metadata include information specific to a station including the station name, station id, latitude, longitude, elevation, and recording date. Errors in the metadata of a station are common and could result in serious problems in a combined data set. Therefore, a check for consistency of variables pertaining to each station is necessary to ensure the accuracy and quality of precipitation data.

The data acquired from the five different vendors were recorded in different formats, thus reformatting data was undertaken. Data from all stations were reformatted into an input format. The reformatting included: unifying the data type (i.e. accumulative vs. incremental precipitation), unifying data units (i.e. inches of precipitation to hundredths), unifying time (i.e. time zones), unifying symbols (number vs. string for missing values), and unifying date formats. It is noted that, according to the Uniform Time Act of 1966, daylight saving time begins on the last Sunday in April at 0200 (2:00 a.m.) local standard time. That time was amended by Public Law 99-359 (July 8, 1986) to the first Sunday in April, beginning in 1987. Therefore, 0201 local standard time would become 0301 local daylight saving time. Standard time begins on the last Sunday in October at 0200 local daylight saving time. Therefore, 0201 local daylight saving time becomes 0101 local standard time. Moreover, the time zone employed between vendors also differed and included Pacific Standard Time (PST) vs. Greenwich Mean Time (GMT); as California is located in PST, the PST was selected as the database time zone. All character strings were also deleted and replaced by numbers. Finally, all dates were reformatted into a common scheme in the database.

Some vendor networks collected accumulative precipitation data (i.e. a running total throughout an event) while others collected incremental data (i.e. accumulation throughout an hour). These different data types were unified into incremental hourly precipitation data since the research interest was in incremental data equal to or greater than 0.01 inch/hour.

To facilitate inter-station comparisons, a table of neighboring stations was developed. This table identified stations that were within 15 miles of one another and whose elevations were similar. The table was generated to support future data processing where the need arose to quickly extract a file from a neighboring target station when data were further quality-controlled.

Precipitation contains three major types: rainfall, snowfall, and mixture of both. While California has a lot of desert areas, snowfall is not uncommon in the high county: the whole Truckee/Lake Tahoe/Reno/Carson City area in Northern California, where snow may be encountered any time between early November and late May. Therefore, this study selected only precipitation data characterizing rainfall, according to the average air temperature of each month. This was done by setting up a threshold air temperature value (based on the average of monthly temperatures). Precipitation above the threshold value was identified as rainfall; otherwise, it is identified as snowfall.

Level-I quality control checks. Level-I QC checks are preliminary error checks that can be performed on individual observations. If an observation failed the preliminary checks, it may be eliminated from the database or manually corrected, depending on conditions and researchers' judgment.

Gross limit check: The historical extreme hourly rainfall in California is 4.7 inches (http://ggweather.com/sf/extremes.html), thus it is reasonable to expect that hourly precipitation totals should be greater than or equal to zero but less than or equal to 5 inches per hour. As a result, any precipitation in the database that had a negative value or unusually large values were flagged and checked further, unless the negative values occurred during the resetting time. Some field measuring devices accumulate precipitation cumulatively during the year. Generally, this sensor type is used for real-time collection duration of hourly or event data. These stations' accumulation tanks periodically dump the accumulated precipitation to make room for more precipitation. This may cause the value transmitted to jump backward several inches. As a result, the cumulative value usually gets larger until it is reset. A reset may occur if a technician visits the site or it is near the beginning of the season, which varies according to different agencies (i.e., July-June, October-September). Note that due to evaporative losses, at some times hourly rainfall total will be a negative number (-1 in/hr < 0 in/hr). In such cases, these values were manually checked to see whether the flagging was legitimate. If the negative value was greater than -1, it was flagged as questionable data. Extreme observations above or below these thresholds were extracted for further quality control.

Duplicate records check: Multiple rows of identical records in which all variables shared the same values in the dataset were considered to be duplicates. In such cases, only one record was kept.

Extra constant records check: If an instrument malfunctioned, it is likely that the data would be constant for a long time period (with the exception of zero values over dry periods). Non-zero values that were constant for over 48 hours were flagged and checked. Neighboring station data were used to provide the spatial comparison for the station in question during the time period of interest.

Level-II quality control checks. Spatial consistency check: The spatial consistency check was used to identify outliers that were not spatially consistent with the neighboring gauges (Kondragunta, 2001). To use this technique, the following steps were undertaken. First, a list of neighboring stations within 0.25 degrees latitude and longitude was generated. Then the data from neighboring stations was compared using this database. Hourly precipitation data were deemed as outliers if they were greater than 2.2 times the standard deviation of all the values from the neighboring stations for that hour. In such cases, the value was flagged. If the hourly precipitation totals at one station indicated a heavy rainfall (>0.1 in/hr) during a short time period when the other neighbor values were all zero, the record was also flagged for further checking. Finally, hourly precipitation totals of zero for a long time period compared to neighboring values reporting heavy rainfall were flagged.

Additionally, monthly/daily rainfall time series around a suspect station were checked and compared to the time series of its neighborhood stations. If the data from a station were erratic or inconsistent with the data from another nearby station, the erratic data were also removed from the dataset. However, the data from its nearby stations were kept in the database. Each annual maximum value was checked by comparing their values with those from the nearby stations (unless the gauge was under the influence of an intense thunderstorm).

Wet hour frequency data spatial consistency check: As this research focused on the number of measurable rainfall hours (>=0.01 in/hr) occurring during each year for each station, the time series of monthly/yearly wet hour frequency of all nearby stations were compared to detect spatial inconsistency. Figure 3 provides an example of a spatial consistency check for nearby stations. The time series plot of monthly total rainfall of four nearby stations was used to determine any significant discrepancy among all these stations' data. The monthly total rainfall of nearby stations was calculated with the hourly total rainfall data for the months from April through October of 2005. Station PR3 displayed an unusually higher trend for April and May than neighboring stations. As a result, the data from these two months at station PR3 were flagged. Furthermore, the monthly wet percent data of these nearby stations from April to October were compared, with no significantly different trends being found (no significant discrepancy among data from the stations). This comparison is presented in Figure 4.

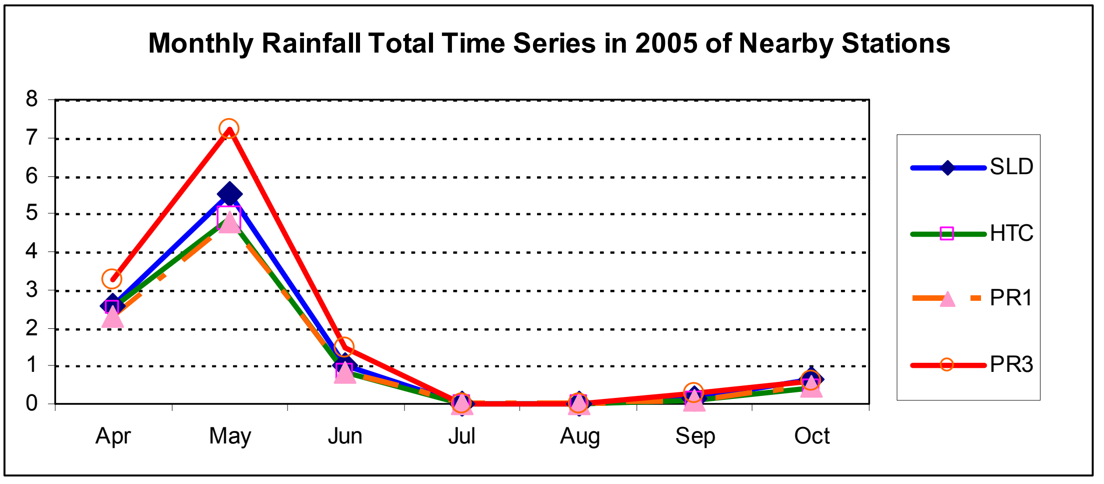

Figure 3. Time series plot of summer monthly total precipitation.

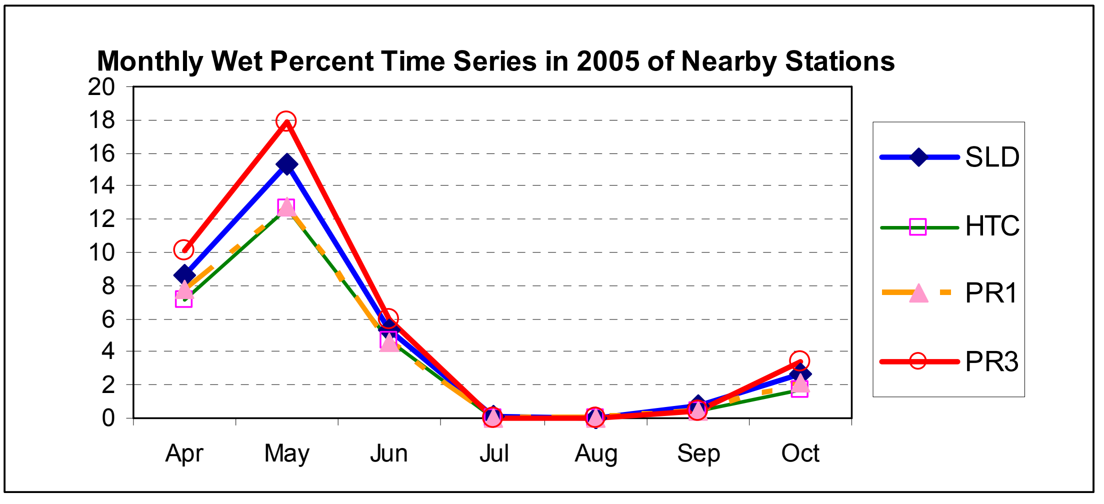

Figure 4. Time series plot of summer monthly wet percent variations.

<u>Temporal consistency check</u>: The temporal consistency check was designed to detect problematic data or stations by looking at continuous station measurements over time. This check was computationally more intensive than the spatial consistency check. The temporal consistency check was performed only if deemed necessary. The process used a 5-year, 31-day window to calculate the mean and standard deviation of the daily precipitation total. This total should fall within the limits of the mean of daily total precipitation by +/- 3 times the standard deviation.

Multiple sensors check: The data from different sources (e.g., vendors) were compared to check for discrepancies. Using daily precipitation total data or monthly average precipitation data, possible errors were identified and flagged, and the extreme-event record was used to identify and confirm the errors.

Level-III quality control checks. Manual quality control check: Data flagged as incorrect or questionable were checked manually by two processes. These included visual expert quality control and interactive correction with graphical assistance (comparisons with climate maps, extreme-event catalogs). Most suspected errors required manual verification and correction. Note that if a gauge was under the influence of an intense storm, it did not have to be spatially consistent with its neighbors. Such gauges identified as outliers from the spatial consistency check were compared against lightning data. If there was at least one lightning strike within a 10km radius of the gauge during the past one hour in question, then the data from that gauge were considered valid.

TREATMENT OF MISSING DATA

The completeness of the precipitation dataset is crucial to determining the wet percent time for each station. In this study, the hourly precipitation data collected were from weather stations reported in the past decades in the state of California. A significant amount of missing data were identified from weather stations that were not operational during certain time periods. In addition, data involving errors in observation reporting and recording or questionable values identified through the Level-I and Level-II QC procedures were treated as missing data. The other missing data occurred when values were reported intermittently for some stations or by sensors. In the above cases, quality improvement of precipitation data should include the treatment of missing data (Figure 1). This section presents the procedures to fill in missing data.

Revision of missing data. If the missing data period was within 48 hours and the values stayed the same before and after the period, we assume that the value should have remained the same and filled in the missing values accordingly. We did not fill in the same values if the period of consecutive missing values was over 48 hours, in order to avoid confusion with a possible “stuck rain gauge”. This missing data revision dramatically reduced the amount of missing values in the dataset. Figure 5 shows the monthly mean percentages of missing data for 482 CDEC stations in 2005, before and after the revision step respectively.

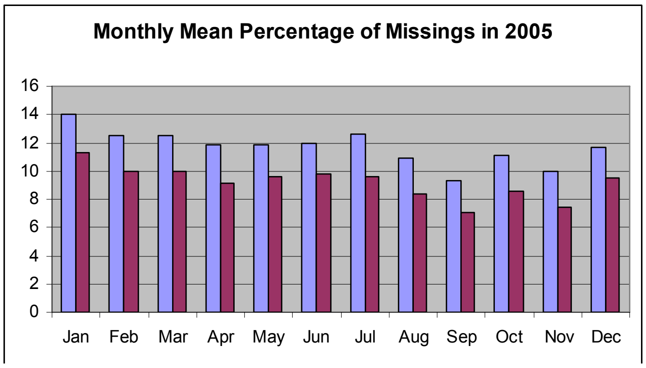

Figure 5. Monthly mean percentage of missing data for 482 CDEC Stations in 2005, with the lighter bars and darker bars representing the archived data and revised data.

The discrete nature of precipitation in time and space has always posed unique problems as compared with more continuous variables such as temperature and pressure. Therefore, estimating missing data of short-time or hourly precipitation has always been a challenge and rarely reported in the literature. General methods for interpolating missing data vary (Cressie, 1993), depending on the number of variables in the datasets. For a multivariate case, single/multiple imputation, maximum likelihood estimate etc. have been used; for a univariate case, list-wise deletion, nearest neighbor regression, ARIMA model, and neural network model have been used (Cressie, 1993). These methods are mostly for data that are normally distributed and spatially and temporally stationary. Because hourly precipitation is essentially a non-stationary process, conventional estimation algorithms for missing data are not adequate (Jin and Wang, 1998).

To fill in missing data, the Hourly Precipitation Frequency Data (HPFD) was derived from hourly precipitation data. Recall that wet percent time was defined as the percentage of hours of measurable rainfall greater than or equal to 0.01 inches during each year. An indicator variable named *wet_flag* was created to indicate when hourly rainfall total fell at or above 0.01 inch. If the hourly rainfall total equaled or exceeded 0.01 inch, the variable *wet_flag* took on the value of 1; otherwise it was assigned a 0. If the raw hourly precipitation total data were missing, the *wet_flag* value was missing as well. As a result, the handling of missing hourly total precipitation data is equivalent to the handling of missing HPFD, which exhibits greater spatial coherence than hourly total precipitation data (Englehart and Douglas, 1985).

Filling in missing hourly precipitation frequency data (HPFD). The Nearest Neighbor Assignment (NNA) method is the simplest and one of the most powerful approaches in data mining (Berry and Linoff, 1997; Han and Kamber, 2001). The missing data from one station are assigned by the observed values of its nearest stations that had data for the hour. Research showed that NNA has comparable performances with other more sophisticated approaches: Pesonen et al. replaced missing data by using NNA in comparison with other three methods (substituting means, random values and neural network) and NNA performed as good as the neural network (Pesonen et al., 1998). Toth et al. developed and compared the accuracy of the short-term rainfall forecast using NNA, artificial neural networks and auto-regressive moving average (ARM) models. The results indicated that NNA performed better than the ARM model, but neural network performed slightly better than NNA (Toth et al., 2000).

In this study, a revised NNA (rNNA) approach is proposed to fill in the missing HPFD. This approach was chosen because the network stations were sufficiently dense and the nearest neighbors were close enough to share the most similar meteorological features that governed rainfall. In addition, there existed the need for handling large volume of data and the need to give timely results (and thus minimize computing).

The rNNA method rules are described as:

- If missing hours fall inside consecutive heavy rainfall (HPD>=1.0in or 25.4 mm/hr), fill in the rainfall values for the missing hours
- If missing hours fall inside consecutive rainfall (0<HPD<1.0in or 25.4mm/hr) and the period lasted for more than 6 hours, fill in rainfall values

Otherwise:

- If only 1 hour of data were missing, use the majority vote method. That is, if the majority of the nearby stations reported rainfall, the missing value was replaced by nearby rainfall value;
- If several consecutive hours (<=24 hr) of data were missing, use the average HPFD of the same time from the nearby stations;
- If several consecutive days (<=7 days) of data were missing, use the average HPFD in the same days from the nearby stations;
- If more than a week but less than a month of data were missing, use the average HPFD in the same month from the nearby stations;
- If more than a month of data were missing, use the average HPFD for each missing month from the nearby stations.
- The percent of missing data should not exceed 15 percent of each year total hourly data.

Validation of the rNNA method. Data sets were extracted from the database to test the validity and applicability of the rNNA method in different hydrological regions. The validation was conducted by creating a sufficient number of gaps of various durations to simulate raw missing frequency data recovery. For instance, we used faked missing precipitation frequency data from a randomly chosen station that had complete data set during a certain month. The error percentage index was calculated after the data were filled in by the rNNA method and time series plots were provided to check for data discrepancy. In the county of Tulare, a station named ATW from CDEC database was chosen to test the validity of rNNA method for missing data with various time periods. The complete data set without any missing values in May 2005 was chosen to evaluate the accuracy of the models after the estimated data were obtained. 384 hours of faked missing data during the month of May were created from the original complete data set. It had a neighboring station of GNF, which was 10 miles away and also had the complete data set at the same month. The rNNA method was used to fill in the missing data of ATW and the summary statistics were calculated before and after the missing data were filled in. An error percentage was also provided and the results indicated that the NNFA (nearest neighbor frequency assignment) method worked well on filling in the missing data. A strong correlation was also found between the data from the two adjacent stations, both in terms of daily total precipitation as well as daily wet percent time in May 2005, after the missing hours were filled in, which demonstrated the validity of the rNNA approach. Details are provided elsewhere (Wang et al., 2008).

Automated and/or manual updating procedure. Due to climate change, it is reasonable to anticipate that in the future, the amount of precipitation an area receives may vary significantly. In response to the climate change, it is necessary to update wet percent time periodically so that most recent weather conditions are taken into account. To do this, all new hourly precipitation data from selected stations will need to be downloaded. This process can be automated by coding. The CDEC, CIMIS, and MESOWEST provide both real-time and archived historical hourly precipitation figures, while the NCDC and NWS only provide historical hourly precipitation data. Associated weather data can be downloaded through the respective sources' wet site download interface.

Once downloaded, all hourly precipitation raw data, which are recorded in different data and time zone formats, need go through metadata checking and data reformatting. Since all the data sources and data formats are fixed, this process can be automated.

The data quality control process presents a challenge in terms of the programming required for automatic updating. As described earlier, three levels of QC need to be followed to ensure the quality and completeness of the collected hourly precipitation data. Currently, the Level-I quality control step can be handled by codes with predefined logic. However, for the Level-II and Level-III QC, manual processing and human intervention are required. These two levels of QC involve visual expert quality control checks and interactive correction with graphical assistance to compare the spatial and temporal distribution of wet percent time with climatic maps, orographical data and extreme-event logs.

Another challenge to automated updating is presented by the process of missing data handling. Significant amounts of hourly precipitation data may be missing from weather stations that were not operational during certain time periods. Additionally, data involving observation errors, reporting and recording errors, or questionable values identified through Level-I and Level-II QC procedures need to be treated as missing data. Other instances of missing data occur when values are reported intermittently for some stations or sensors. The missing data handling process involves revision of such data, as well as employing the rNNA method to fill in missing HPFD. In this study, all of these different types of missing data were handled on a case-by-case basis and filled in either by codes or by hand, depending on the circumstances. As a result, it may not be entirely feasible to automate this entire process, as some human intervention is often required.

DISCUSSION

Given the quantity of data comprising the central data set, some compromises had to be made for the sake of processing time. As the element of interest in this research was hours that received precipitation of at least 0.01 inch/hour and not the actual cumulative amount of precipitation, the data set was not corrected for systematic gauge-measuring errors. Additionally, because rainfall is highly variable in space and time, traditional QC methods were not entirely appropriate for the data used in this research and were revised accordingly for the purpose of estimating wet pavement accident analysis.

The quality and completeness of hourly precipitation data were variable and different data sources had different quality issues. Very limited amount of literature addressed approaches on QC, especially missing data handling for hourly precipitation data in this large quantity. No existing method can be used directly without customization. The missing data handling technique we used has made considerable improvement in the quality and completeness of datasets. The prototype testing has shown that the proposed rNNA method was well-suited to solve most missing data problems. However, further study is required to validate it for data from the stations without closely adjacent nearby stations, for which other approaches may have to be explored.

This research has led to the development of an updated set of wet percent time factors for California highways. These new results were used by Caltrans to replace the old set, which was developed with data that is 40 years old. The updated information was used for the identification of high wet collision concentration locations, which were further investigated individually to evaluate collision risk (Veneziano et al., 2009). Results of McNemar tests indicated that there was no statistical difference between lists produced using a singular wet percent factor and ones produced using finer resolution factors.

This study used substantial amount of weather data from various sources in California. Among the data sources, MESOWEST, NCDC, and NWS are national weather services and provide the majority data for wet percent time estimation. The proposed methodology can be applied to other states that are conducting wet pavement accident analysis. Although other states may have different weather sources, the only difference in applying this methodology would be in the data preprocessing phase. Once raw data are formatted, data quality can be controlled by following the same procedure as the California case study. Note that this brief paper focused on the description of the methods, and more details (e.g., descriptive statistics of the data) are provided elsewhere (Wang et al., 2008).

CONCLUSIONS

Wet percent time and other factors (e.g., wet pavement exposure, WAI) are commonly used by state transportation departments to identify high wet pavement collision locations. QC of precipitation data is essential to obtain accurate wet percent factors and make sure that high risk locations are correctly identified. Based on the comprehensive literature review, this study proposed a methodology for precipitation data QC. This method includes various quality checks as well as treatment of missing data. Data collected from over 2,000 weather stations in California were used to demonstrate this methodology and the results have been used by Caltrans to update its wet percent time data across the state. The methodology can be adopted by other states conducting similar studies to identify high wet collision locations or for applications such as climate change research.

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Extended Performance of Media Filter Drains: New Media

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Abstract

There are numerous linear best management practices used by many state departments of transportation (DOTs) for highway runoff treatment. An example of one type is a media filter drain (MFD) which is a trench filled with aggregate and three active ingredients: dolomite, gypsum, and perlite (MFDs are also called Ecology embankments or bioslopes). Initial research on MFDs predicted that their life spans were at least10 years. Recent studies indicate that for dissolved zinc and copper metal removal, the life span may exceed 20 years. The dissolved concentrations of these metals are of issue as higher concentrations in receiving waters can impact many aquatic species, particularly some cold water fish such as salmon. This study extended the life further in laboratory tests for MFDs made with media with slightly higher amounts of active ingredients. Zinc and copper removal remained high. However, some possible channeling was suspected with slightly lower removal rates after long periods without moisture.

INTRODUCTION

Stormwater is rain and snowmelt that develops into runoff. Prior to infiltrating into the ground or discharging into receiving waters, stormwater can run over rooftops, and through streets, highways, parking lots and other land surfaces, collecting pollutants along the way. This ‘runoff’ can flow into streams and other bodies of water which may be detrimental if untreated. These pollutants can be harmful to plants, animals, and people. Some examples of these are the oils, fertilizers, pesticides, soil, municipal solid waste, and animal waste that might diminish stormwater quality. Pollutants from runoff could include organics, nitrate, phosphorus, total suspended solids, volatile suspended solids, etc. (Barrett et al. 1998). In addition, dissolved metals might be found in stormwater from various surfaces and transportation sources. The main metals of concern for aquatic life are copper and zinc (Scholz et. al. 2011). This research focuses on copper and zinc removal which are pollutants of interest for aquatic life in the Pacific Northwest.

Copper and zinc continue to be the metals identified as major pollutants when discharged to certain receiving waters with sensitive species. Zinc can be found in industrial facilities, galvanized metals, and automobiles. Copper in the same manner can be found in vehicles, particularly from the vehicle exhaust and brake pad wear. When these metals flow to major bodies of water, they are exposed to aquatic life habitats. Pacific salmon have been identified as a species of fish directly affected in coastal watersheds and estuaries (Sandahl et. al. 2007).

Media filter drains (MFDs) are a feasible solution for stormwater quality control from roadways for numerous reasons. First, MFDs take a relatively small width compared to bioretention systems and have high removal rates for dissolved metals. Bioretention cells are described more along the lines of "engineered soil" and most often have a retention system. Bioretention was developed in the early 1990's utilizing soil and herbaceous plants to remove stormwater pollutants. Bioretention systems usually require an overflow, although modifications have been implemented where underdrains can be installed to direct the effluent to existing sewer systems (EPA 199). In comparison, MFDs are drains that detain but do not typically perform extensive retention. MFDs use a rock filled trench with some inorganic active material, and are not intended to mimic a soil/vegetation system. Therefore, they tend to be simpler with less costs and maintenance required, and accordingly are appropriate for many long linear projects without extensive vegetation such as next to a roadway in the right-of way (WSDOT 2014).

MFDs around the state of Washington are typically comprised of four main parts: gravel with no vegetation zone right next to the paved edge, a vegetated filter strip, media filter mix in a trench, and a gravel-filled underdrain. The media filter mix includes aggregate, gypsum, perlite, and dolomite (WSDOT 2014). Constructing media filter drains are considered inexpensive to other solutions and excel in removing higher concentrations of copper and zinc. Field studies utilizing simulated storm events have shown removal rates of 98% consistently for zinc and copper, while nitrogen and phosphorus varied depending on the season (Hatt et. al. 2009). Zinc and copper uptake is probably due to complexation with the ligands in the active media (carbonates, hydrates and sulfates) and ion exchange with the calcium and magnesium ions (Stumm and Morgan 1996). According to an initial study by the Washington State Department of Transportation (WSDOT), MFDs have a lifespan of at least 10 years (WSDOT 2006).

Previous research done by Washington State University (WSU) tested MFDs located in Western Washington to see if they were effective after 10 years of use (Thomas 2013). At the same time, new media was also used in the WSU laboratory studies comprising of the same content but with less fines in the aggregate. This previous investigation was done in order to determine if readily available aggregates from typical highway jobs (which have fewer fines) would work as effective as the original media mix design. This new media mix design is an economical solution since the aggregate gradations do not have to be specially made. This previous research found the new media mix design to be highly effective for copper and zinc removal, with lifespans of many years (Freimund 2013, Freimund et al. 2015). To test this effectiveness and longevity, the media was aged with simulated storms, which were also known as events. Each event constituted a certain fraction of a year for copper and zinc loading based upon conditions typical to Western Washington. The objectives of this current study were to extend the simulated lifespan on some columns from the Freimund (2013) study and to also vary the flow rate into the media in order to see if the MFDs might be effective during extreme storms.

METHODS

As previously mentioned, there were two objectives for this current research. First was to extend the laboratory simulations from earlier new media tests. Second was to vary the infiltration rate to determine the efficacy of the filter drains in intense storms.

In order to test MFDs, four identical columns from the previous study were used for this current study. All 4 columns were 152 mm (6 inches) wide and composed of 305 mm (12 inches) depth of new mixed media in a plastic cylinder. All 4 columns in the previous research, up to Event 112 had previously been loaded with zinc and copper in the aforementioned research to initially evaluate lifespan (Freimund 2013). In the events, these columns were loaded with simulated stormwater that had dissolved copper and zinc concentrations well above any found in nonindustrial stormwater. In this current research, these columns were loaded with additional events up to Event 158, where each loading was referred to as an event and simulated metal masses similar to a large storm with concentrated runoff entering the system. This methodology follows the steps from Freimund (2013) and is herein referred to as accelerated loading so that many years of use could be mimicked in the laboratory in only a few months, except that the loading rates for this current research varied for the different events. A summary of the methods follows.

Simulated stormwater for the accelerated events was comprised of synthetic rainwater solution and metal stock solution. Dr. Marcus Flury of Washington State University provided the synthetic rainwater formula from rain samples taken near the Hanford site in Washington. Synthetic rainwater was made by adding to one liter of deionized water the following chemicals:

- 0.002472 g sodium chloride
- 0.000336 g sodium bicarbonate
- 0.003932 g potassium nitrate
- 0.0003 g potassium bicarbonate and
- 0.0003 g calcium carbonate.

Then 696.4 mL of this rainwater solution was mixed with 77.4 mL of a metal stock solution which was used for a single loading event. (The metal stock solution had concentrations of 40 mg/L of zinc and 8 mg/L of copper.) Note that typical highway concentrations are around 100 ppb zinc and 20 ppb copper, whereas the equivalent concentrations for these accelerated simulations were around 3900 ppb zinc and 650 ppb copper.

Figure 1. Laboratory setup for accelerated loading.

For each event, each column had a different loading rate. Solutions were poured at a rate of approximately 250, 510, 850 and 1270 mm per hour (10, 20, 33.3, and 50 inches per hour) using a separatory funnel (See Figure 1) into each column. These are areal loading rates, representative of a combination of runon and rain surface loading for MFDs for very large storms for various additional volumes of runoff from the roadway. For every loading session, this constituted one event. This process was repeated approximately every 48 hours to simulate wet then dry periods in Western Washington. A total of 46 events were performed on each column. For Events 113 through 132 the loading rate was 250 mm (10 inches) per hour as in the earlier studies. Then the loading rates varied. From Event 133 and on, after every 3 events, the columns would rotate in a cycle and were individually loaded with a different loading rate, again approximately 250, 510, 850 and 1270 mm per hour (10, 20, 33.3, or 50 inches per hour). The rotation was done to decrease column specific variability and represent a real life approach.

Prior to starting an event, a sample of influent was collected in 15 mL vials. The separatory funnel was opened and the event was timed. Effluent was collected in another 2000 mL beaker located under the column. Timings were recorded when the separatory funnel became empty to calculate loading rate. The columns were allowed to drip for an additional 10 minutes after the funnel was empty when collecting the effluent. Effluent samples were then collected in 15 mL vials. A low pH was necessary to keep zinc and copper dissolved until the time of analysis. To maintain low pH levels, 23 microliters of nitric acid were added to both the influent and effluent vials (Eaton and Franson 2005). Lab analysis was performed at the Washington State University's Geoanalytical Laboratories on these vials for dissolved copper and zinc.

RESULTS

Extended Longevity. The first objective was to test the longevity of new media. To calculate this, 46 more events were performed using the columns from previous new media tests. From this data, measures of reduced concentrations show that copper removal ranged between 90-99% while zinc removal ranged between 93-99% for Events 25-158. Figures 2 and 3 are the results for these extended longevity tests which indicate that the media filters are stable and fully functional even after more than 18 years of service as estimated from the previous new media tests (Freimund 2013).

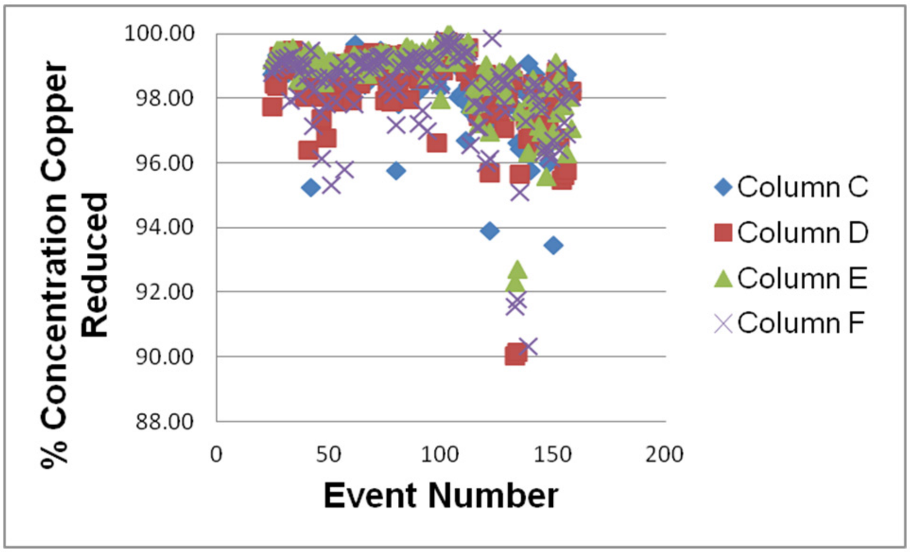

Figure 2. Concentration of copper reduced for columns c-f (events 113 through 158 are from this additional research effort).

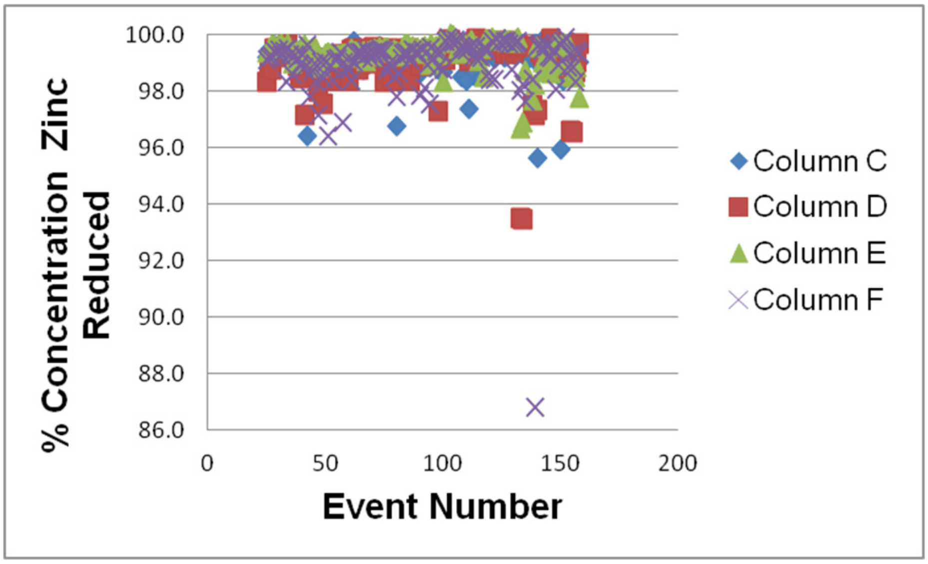

Figure 3. Concentration of zinc reduced for columns c-f (events 113 through 158 are from this additional research effort).

Note that regardless of the influent concentrations well above any found in roadway stormwater, and the variability of the loading rates in the events post Event 113, the removal rates were impressive and continue to remain high beyond the previous testing. Some of the variability post Event 132 was probably due to the much higher flowthrough rates used. In addition, a three month hiatus was taken between Events 112 and 113 and the additional dryness of the columns post Event 112 may have resulted in shrinkage and channeling.

Based on the number of events that were performed, an estimate for the age of the media filter drain could be determined. The same estimation process was used even when varying loading rate speeds. Again, the methodology was consistent with the aging estimation from Freimund (2013). The lifespan according to Tables 1 and 2 are based on the rainfall of approximately 1.02 m (40) inches per year. It was also considered that the runon areas were 9 times the surface area of the media filter drains used. Therefore, for each year of aging there would be 10.2 m (400 inches) per year loaded at the typical concentrations. Typical concentrations on highway sites in Washington for the purposes of these calculations were set at 100 parts per billion of zinc and 20 parts per billion copper. The aging process for zinc and copper varied due to the concentration levels used for the tests. Dividing the 10.2 m (400 inches) per year by the ratio of the accelerated aging concentration (as actually loaded onto the column) by the typical concentrations assumed, provided the researchers with an estimate of the fraction of a year of metal loading for each event. These were calculated separately for zinc and copper, and then summed to estimate the accelerated age. For other applications, these age estimates can be modified by substituting the local typical concentrations, annual rainfall rate and extent of runon.

Table 1. Metal Loading Equivalent Years for Zinc.

		Estimated Media Age at Event	
Columns	**Starting Age**	Event 112	Event 158
C, D, E, F	0	18.2 years	25.7 years

Table 2. Metal Loading Equivalent Years for Copper.

		Estimated Media Age at Event	
Columns	**Starting Age**	Event 112	Event 158
C, D, E, F	0	15.2 years	21.4 years

Variable Loading Rates. The second objective was to explore variable loading rates for Events 113 through 158. The standard loading rate in the previous testing through Event 112 was approximately 250 mm/hr (10 in/hr). Higher loading rates in this research were used on many occasions to investigate the effect of high flow-through on the filters. The results for Events 113 through 158 are plotted in Figure 4 for copper and Figure 5 for zinc, with loading rates grouped as shown. As can be seen, the very fast loading rates tended to have higher effluent concentrations, although removals remained high.

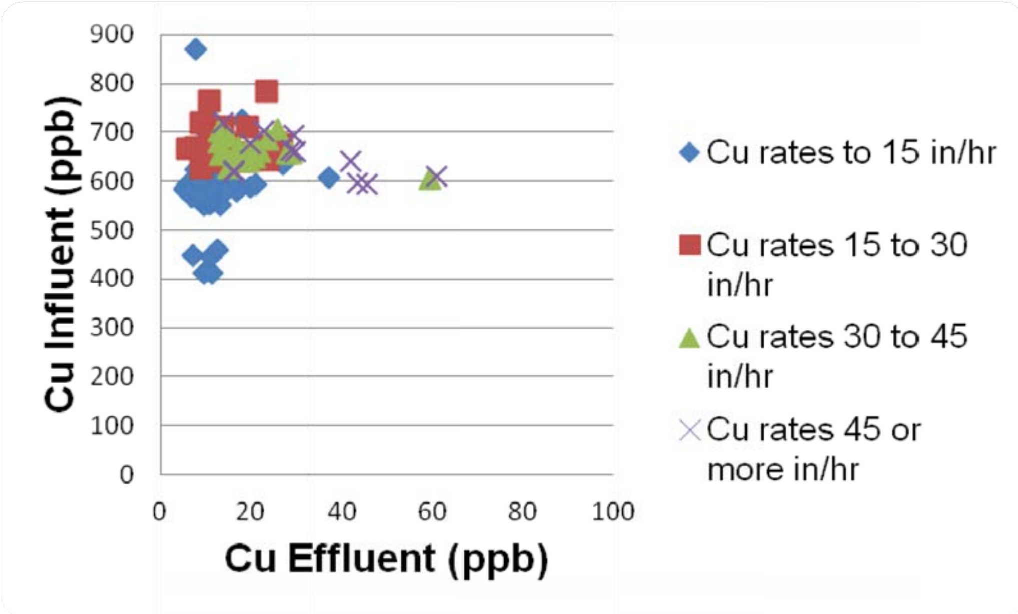

Figure 4. Copper influent vs. effluent based on categorical loading rates (events 113 through 158). note 1 in/hr = 25 mm/hr.

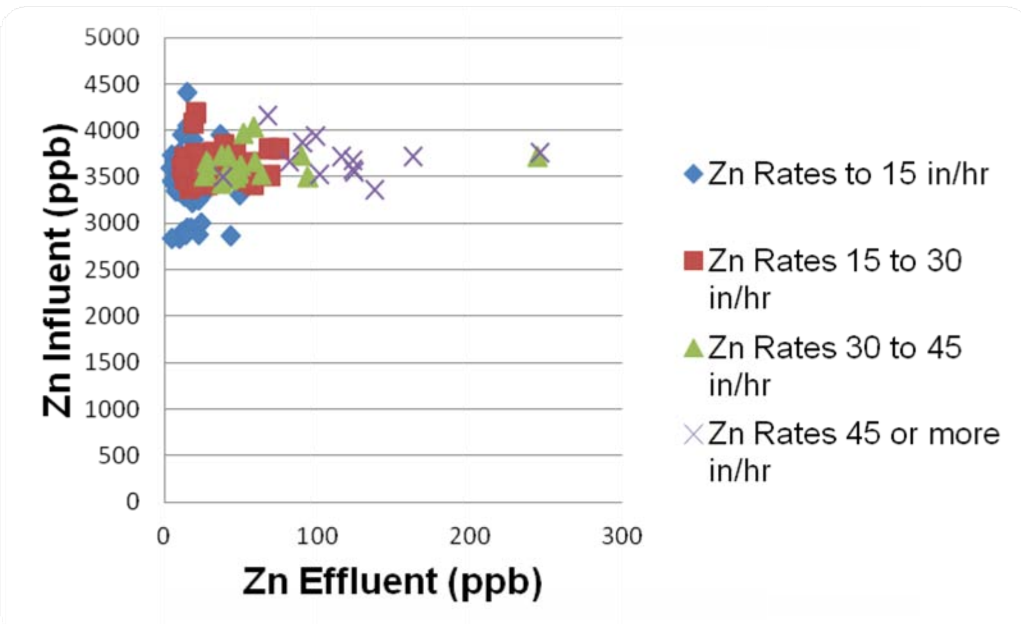

Figure 5. Zinc influent vs. effluent based on categorical loading rates (events 113 through 158). note 1 in/hr = 25 mm/hr.

CONCLUSION

According to the study, it has been determined that the lifespan of media filter drains can extend at least to 21 years based on typical rainfall and runoff calculations in Western Washington, surpassing the original 10 years proposed. It must also be noted that the surrounding environment that media filter drains can be place in have multiple variables which can modify the estimations of longevity. This includes runon areas, highway lanes, and rainfall intensities and volumes. For example, if the media filter drain was placed next to a single lane highway, there could potentially be half the estimated amount of runon leading to double the expected lifespan as estimated herein.

The results in Figures 4 and 5 are based on metal influent concentrations well above any concentrations found in typical roadway applications. These values may be more indicative of industrial facilities, and are unlikely to be found in highways. As illustrated, high removal rates were still achieved even with concentrated stormwater solutions with rapid loading rates. Further studies can be performed to further investigate removal rates at lower concentration levels while maintaining fast flow through. This would be especially useful for evaluating expected performance after the accelerated aging.

The experiments were based on locations around Washington, where typical media filter depths are 305 mm (12 inches). Based on the parameters associated with these removal rates, the results might be useful for designing applications with deeper or shallower media mixes. Based on accelerated loading, it is proven that the new design for media mix appears to be a very effective treatment system for zinc and copper for decades. Since only laboratory work was performed, future studies should extend to field applications. In addition, regional variability based on climate such as after long periods of droughts should be investigated as evidenced by the slight reduction in efficiency when the media was dried out.

ACKNOWLEDGEMENTS

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Tack Coat's Vital Role in Assuring Optimal Flexible Pavement Performance

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Abstract

Tack coats are a vital component of an asphalt pavement's structural system because they bond the multiple asphalt lifts into one monolithic layer. Poor tack coat application results in poor layer bonding. The usual pavement distresses associated with inferior bonding are slippage cracks and shoving along with delamination of the surface lift. Often poor tack coat also contributes to more classic structural pavement distresses, namely fatigue cracking and potholes. Researchers have indicated that even with a loss of only 10% of bond strength, fatigue life can be reduced by as much as 50%. Moreover, the cost to an agency in the event of a bonding failure can potentially exceed the original costs of a maintenance overlay. Despite these facts, little attention is often paid to tack coat operations by both contractors and agencies, and pavement performance suffers.

INTRODUCTION

Goals of highway agencies since they began to build and maintain highways has been the construction of smooth, quiet, long-lasting pavements. The pavement of choice in trying to accomplish these goals has been flexible asphalt pavements. A very key, but often overlooked, component of an asphalt pavement is the bond or tack coat. Tack coat is a sprayed application of an asphalt binder upon an existing asphalt or Portland cement concrete pavement prior to an overlay, or between layers of fresh asphalt concrete. This thin membrane of asphalt binder provides the glue between the layers creating a monolithic material which works as a unit as opposed to unbound, independent, layers. When a pavement is thusly built, it will provide the desired characteristics for its users, while meeting the needs of an agency for an economical, environmentally friendly and sustainable material. Poor bonding of a pavement surface layer has long been associated with inadequate tack coat practices resulting in slippage and shoving of the pavement as seen in Figure 1. This type of failure is most frequently seen in locations where breaking or acceleration are common such as intersections. As will be seen, other distresses can also be made manifest due to poor tack coat bonding, most notably fatigue.

Figure 1. Slippage failure due to poor bonding.

Common materials used for tack coats are asphalt emulsions and paving grade binders. By far the most common choice are emulsions with paving grades the next most common. While cutbacks are still used, their usage is so much less than the other options, they will not be address further in this paper. Moreover, research has shown that cutbacks achieve lower bond strengths then other options, (Mohammad et al. 2012)

TACK COAT DEFINITIONS

It is important for all those involved to be speaking the same language when addressing tack coats. Therefore the following definitions are offered:

Tack Coat—sprayed application of asphalt cement upon an existing asphalt or Portland cement concrete pavement prior to an overlay, or between layers of fresh asphalt concrete.

Original Emulsion—an undiluted emulsion which consists primarily of a paving grade binder, water, and an emulsifying agent.

Diluted Emulsion—an emulsion that has additional water beyond the original emulsion added to it.

Residual Asphalt—the remaining asphalt after an emulsion has set typically 57-70 percent of the original emulsion.

Tack Coat Break—the moment when water separates enough from the asphalt showing a color change from brown to black.

Tack Coat Set—when all the water has evaporated, leaving only the residual asphalt. Some refer to this as completely broken.

STRUCTURAL DESIGN

Pavement engineers design pavement structures based on a series of inputs. These inputs include the design life of the pavement; engineering properties of the materials; and the expected traffic over the life of the structure. The typical design life for an asphalt pavement is 20 years. However, increasingly engineers are seeking to extend the design period and are employing the principles of perpetual pavements. Perpetual pavements are thicker initially than traditional counterparts, but they are expected to perform for at least 50 years without deep structural maintenance or rehabilitation. Distresses in a perpetual pavement are expected to be confined to the uppermost layers thus requiring periodic milling off and replacement of the driving surface while retaining the underlying materials. Since 2001 the Asphalt Pavement Alliance has been issuing Perpetual Pavement Awards to primarily state departments of transportation. Thus far 93 such awards have been granted to 29 states (APA 2015).

Regardless of the methodology employed, a common assumption of all thickness design methods is that the pavement layers will be working together as a single monolithic unit. When an asphalt pavement is built in this way principal failure mechanism are bottom up cracking stemming from tensile strain at the bottom of the pavement layers, or rutting from compression of the subgrade material in typical pavement structures. If the pavement was designed and built as a perpetual pavement, principal failure mechanism will be top down cracking at the surface, typically confined to the top layers. If layer bonding was not achieved, neither traditionally designed pavements nor perpetual pavements will perform as intended. Within a debonded pavement the strain profile will not match that of an equally thick, but properly bonded pavement. This will lead to decreased fatigue life as the strains at the bottom of the debonded layer will quickly become excessive which can result in fatigue cracking initiating at the bottom of a layer within the asphalt.

LITERATURE REVIEW

Failure to bond pavement layers is known to result in sliding and shoving of surface layers of pavement. However, a reduction in fatigue life is also a very probable consequence of poor bonding as well. When a pavement is not properly bonded and the layers exhibit independence, an alteration to the stress profile will result. A variety of researchers have reported on this situation over the years.

Roffe and Chaigon (2002) reported that if a pavement displayed no bonding within its layers, a 60% loss of life could be expected. Similarly, Brown and Brunton (1984) reported that no bonding would cause a 75% reduction in pavement life, and at 70% bond strength, a 70% reduction in pavement life could occur. Moreover, May and King (2003) reported that with only a 10% loss of bond, fully a 50% reduction in fatigue life would be expected.

While all of these examples drew their conclusions based on computer models of pavement behavior, an extensive analysis was performed by Willis and Timm (2006) of a bond failure at the National Center for Asphalt Technology's (NCAT) Test Track in Alabama. The failure analyses occurred during the 2003 test cycle in which two sections (N7 and N8) were constructed to test the ability of a rich bottom layer (RBL), one (N8) had 0.5% added binder above its determined optimal value, to better resist fatigue cracking versus a traditional hot mix asphalt (HMA) layer. All other aspects of N7 and N8 were identical including instrumentation at depths of 125 and 175 mm and a total asphalt thickness of 175 mm for both. Surprisingly, N8 exhibited fatigue failure far sooner than N7 prompting a full forensic investigation.

Investigators utilized a multifaceted approach as they attempted to ascertain an explanation for N8's poor performance. One tool they used to analyze both sections was the strain data supplied by the in-pavement instrumentation. Secondly, they utilized the WESLEA Windows-based, linear elastic program to calculate theoretical strain values for various scenarios of bonding and debonding. A comparison of these first two analyses was made. Trenching was also performed to physically investigate the two sections. And lastly, cores were extracted from selected locations in each section for bond testing.

The results of this extensive analysis of the two sections lead NCAT researchers to conclude that a debonding had occurred between two of the layers within section N8. In fact, the measured strains matched those predicted by WESLEA very well. The two slippage locations were between the SMA surface and the HMA under it as well as between the RBL layer and the HMA above it. Moreover, they determined that fatigue cracking had initiated in the HMA at the interface, and above the RBL. This cracking prompted the early fatigue failure reported in N8 as it was unable to behave as a monolithic unit as designers had intended.

In the state of Missouri (Jason Blomberg 2014) it was reported that an Interstate pavement experienced fatigue cracking early in its life, after 8-10 years of service, Figure 2. MODOT conducted a forensic analysis which included the collection of cores. As can be seen in the Figure 3, bonding failure was found at various locations with in this structure. Again, non-monolithic pavements will exhibit compromised performance. It is reasonable to question how many pavements that display premature fatigue are the result of poor bonding and not other potential causes.

Perhaps the most definitive research effort on tack or bond coats to date was National Cooperative Highway Research Project (NCHRP) 9-40. As reported in NCHRP Report 712, the research team tested the bond strengths of a variety of tack coat materials including emulsions, cutbacks and paving grade binders; various residual application rates from no tack coat to 0.702 liters per meter squared (l/M^2); various surface types including old hot-mix asphalt (HMA), new HMA, milled HMA, and grooved Portland cement concrete (PCC); and test temperatures from -10 to 60C in 10 degree increments; in shear, tension, and torsional configurations. Test specimens were obtained from both field and laboratory produced materials. Additionally, results from an extensive international and domestic survey of their standard methods and materials was reported.

Some of the recommendations from Report 712 include residual tack coat application rates for different surface conditions; shear bond strength testing; milling of existing surfaces to improve performance; and that current typical application rates may be too low. A follow-up study, NCHRP 9-40a, is underway applying results from 9-40 to field locations in a variety of states.

Figure 2. Fatigue cracking on missouri pavement.

Figure 3. Cores showing layer debonding.

TACK COAT COSTS

To determine the real costs associated with the use of tack coats, a variety of 2013 bid tabs were collected from various states that pay for tack coat separately and do not treat it as an incidental paving item. Two types of projects were examined: mill and overlay and new or reconstruction. For each type of project, the value bid by the winning contractor for the tack coat material relative to the total cost of the project as a whole and relative to the total pavement costs were determined. Once this information was calculated, it was then assumed that a bonding failure had occurred between the top lift and the layer below it. A second analysis was then performed which assumed the following. The bond failure was such that all of the top lift was in need of full removal. This material would be milled off and replaced with a new lift identical in thickness to that removed. All associated project costs (milling, materials, traffic control, pavement markings, etc.) would be identical to those found on the bid tabs.

The above described analysis resulted in the following conclusions. The cost of tack coats on new or reconstructed facilities was 0.1-0.2% of the total project costs. On the mill and overlay projects, tack coat was 1.0-2-0% of those project costs. If a bond failure occurred and the remedial action described above were performed, the cost to replace just the top lift was found to be between 30-100% of the original project costs. The lower end of the range was for a new or reconstructed pavements which consist of multiple lifts, and the higher end was for a pavement that had seen a mill and overlay of just a single lift of material.

TACK COAT BEST PRACTICES

As has been demonstrated, a well bonded pavement functioning as single unit is essential to achieve performance expectations. To produce the best opportunity for a pavement to achieve its performance expectation, good design and construction practices need to be followed.

The design phase for tack coats involves and evaluation of the surface to which it will be applied, selection of an appropriate tack coat material, and selection of the proper residual asphalt rate are all required.

When evaluating the surface, designers should consider the following characteristics by answering certain questions. Is the pavement a new or an existing facility? If the pavement is existing, will it be milled or not? If it is not going to be milled, then how weathered, raveled, or worn is it? In general, the more surface area the pavement has, the greater the volume of tack coat needed for optimal bonding. The Asphalt Institute offers the following general recommendations for tack coat residual rates on different common surfaces as seen in Table 1. The Institute encourages applications tend toward the upper ends of these ranges for best performance. It is further encouraged that all surfaces receive an application of tack coat.

The selection of the appropriate tack coat material is based on a combination of material properties and availability. The most common tack coat materials are asphalt emulsions with slow setting emulsions being the most common of the emulsions. Emulsified options are growing with a trend towards stiffer asphalt base binders. Some of the advantages emulsions offer is that they lend themselves to application uniformity; there are typically numerous choices in most markets; and contractors are familiar with their usage. Disadvantages include the time it takes for the emulsion to break and the potential for tracking of emulsified materials. The afore mentioned trend towards stiffer base binders is largely in an effort to combat tracking as emulsions formulated with a stiffer base binder are less prone to tracking. Excellent performance has been realized when asphalt emulsions are properly used.

Table 1. Recommended Tack Coat Residual Asphalt Rates.

Surface Type	*Residual Application Rate Liters/Meter^2*
New Asphalt	0.091-0.158
Existing Asphalt	0.181-0.317
Milled Surface	0.181-0.363
Portland Cement Concrete	0.136-0.226

Other materials that are specified include both paving grade binders and cutback asphalts. Paving grades are most commonly used in southern states although not exclusively. Nighttime paving projects also employ them as they do not have a break or a set time due to the lack of water within them. Excellent performance has been reported when using paving grade binders as well.

Surface preparation is vital to give the materials the best opportunity to achieve a high bond strength. The goal of surface preparation is to produce a clean, dry surface. On existing pavements, milling is encouraged as its benefits are many. First, milling removes the uppermost materials which is typically the most compromised by traffic wear and weathering. Second, milling helps to smooth out any irregularities in grade than may have developed within the pavement. And third, milling improves the bonding characteristics of the overlay to the existing pavement. The negative aspects of milling are the cost associated with milling and the added cleaning typically connected to milling as it can produce a fair amount of debris. Milling may also increases the amount of tack coat needed as it increases the surface area of the existing pavement. However, it is this additional surface area along with an increase in aggregate interlock that promote the improved bonding characteristics seen with milling.

On new or reconstructed pavements, or where multiple lifts are a requirement of construction, surface preparation between lifts is generally minimal. Cleaning may be the only need on new surfaces receiving another lift of asphalt. It is not uncommon that no surface cleaning is needed. However, if the freshly laid pavement has become dirty it behooves the contractor to clean any and all such locations prior to the next lift being tacked and paved.

Once the surface has been properly prepared, application of tack coat can proceed. Tack coat applications should be uniform and consistent both transversely and longitudinally. An example of inconsistency in both directions can be seen in Figure 4. Unfortunately it is not uncommon to see tack coat applications that are streaky or striped in appearance. Some refer to this as “zebra tack” (as seen in Figure

5) and it does not produce good bond strengths. Recall that some researchers reported that a mere 10% loss in bond strength resulted in a 50% loss in fatigue life.

Figure 4. Inconsistent tack application.

Figure 5. "Zebra tack".

An example of exemplary contractor consciousness is seen in Figure 6. Here a contractor applied a very uniform tack coat to most of the milled surface. Unfortunately they missed one narrow section. However, instead of ignoring this miss and considering it "good enough", they applied material to the missed section. All surfaces should be tacked with special attention paid to any longitudinal joints to help assure strong performance in this vital area.

Figure 6. Contractor applying material to area originally missed.

However, Tack coat dilution needs to be addressed. Historically emulsified tack coats were diluted to assist contractors in achieving a uniform application of tack. This was a virtual requirement due to equipment limitations found in the past. However, today's modern distributor trucks are fully capable of applying a uniform tack coat without dilution. Further, careful control is needed to properly account for any water added by dilution so that the residual application rate can be calculated. Without such control, residual application rates are impossible to determine. Therefore, dilution is not recommended. Furthermore, if dilution is going to be allowed, it is recommended that it only be performed by the supplier where a greater degree of control can be expected rather than in the field.

Calculating residual asphalt application rates need to account for not only the water that is present in an original emulsion, but also any added water via dilution. For example, if an application rate of 0.45 l/M^2 was applied of an emulsion diluted at 70:30 (original emulsion:water), and the original emulsion contained 33% water, calculation of the residual shot rate would need to account for both sources of water. Thus, the 0.45 l/M^2 would be multiplied by 0.70, to account for the dilution, and 0.67, to account for the water in the original emulsion. Therefore, the residual tack coat rate in this example would be 0.22 l/M^2.

Distributor truck setup is also vital to the application of a proper tack coat. With the various asphaltic products that can be applied with a distributor truck, different nozzles are available to best match the material with the application rate. Consultation with the manufacturer of the distributor truck should occur to ensure that the most appropriate nozzle is installed for the tack coat material being applied. While modern distributor trucks have excellent capabilities and they are typically computerized to help minimize errors, calibration of the truck is still needed to further verify residual application amounts.

Nozzles also need to be aligned properly. This requires a 15-30° offset from the spray bar as seen in Figure 7. The offset prevents the fan from one nozzle from interfering with the fan from another, thus improving uniformity of application.

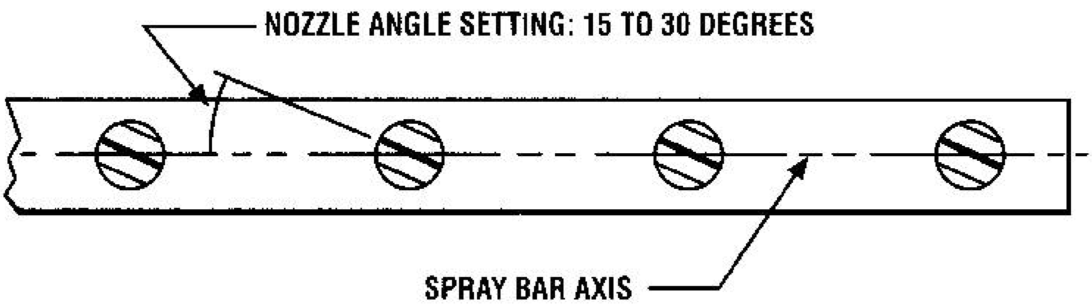

Figure 7. Proper spray bar nozzle orientation.

Proper spray bar height is also important. Setting this height to provide a double, or preferably, triple overlap as seen in Figure 8 is also crucial to achieving application uniformity. Moreover, this overlap helps to maintain coverage if any of the nozzles were to become compromised during the application. Triple coverage is maintained with a spray bar height of about 305 mm.

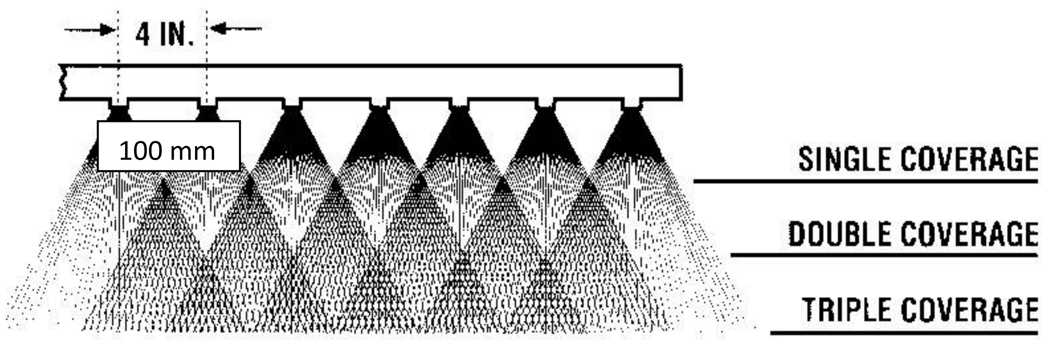

Figure 8. Display of Single, Double, and Triple Overlay Coverage.

CONCLUSIONS

Bonding of pavement layers is vital to the creation of long life asphalt pavements. With proper bonding of the layers, a monolithic structure is formed greatly improving a pavement's resistance to strains and fatigue. This configuration is consistent with the assumptions common to all pavement thickness design methods. Failure to achieve adequate bond strength results in increased potential for pavement layer slippage, shoving, and/or fatigue cracking. Moreover, the cost of tack coats relative to project costs are minor. But the cost of a bond failure can quickly escalate to where it could potentially match the cost of the original project. It is, therefore, most advantageous to properly select and install tack coats so that overall pavement costs are minimized, while environmental stewardship is maximized.

Selection of an appropriate tack coat material, applied in the recommended ranges provides the glue necessary to the pavement for bonding. Surface preparation creating clean and dry material is desirous for bonding. Milling of exiting surface materials will further improve bonding capabilities, thus typically improving pavement performance. Setting up, maintaining, and calibrating the distributor truck is also needed to provide the desired uniform application. Selection of the most appropriate nozzles to match the materials and the target residual rate needs to be made in reference to the distributor truck with the spray bar set to either a double or triple overlap configuration.

In an increasingly environmentally conscientious world, in conjunction with ever tighter agency budgets, maximizing a pavement's life has never been more wanted. One of the more minor, but absolutely vital, components of pavement is its tack coat. Poor performance by this minor component can results in major failure of the pavement. This unnecessarily taxes an agency's budget, disrupts the drivers, and wastes natural resources.

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Managing Metallic Corrosion on Winter Maintenance Equipment Assets

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Abstract

Many metallic components in highway maintenance equipment fleet are at risk of corrosion, which is exacerbated in service environments where roadway deicers have been applied. Roadway deicers generally feature chloride salt(s) as their freezing point depressant. As such, the exposure to roadway deicers poses a substantial risk to highway maintenance equipment. This work will review best practices available to manage the risk of deicer corrosion to equipment assets and present a user's manual for the use of maintenance agencies in the northern climate. The manual has addressed the following subjects: Corrosion Basics; Causes and Effects of Corrosion; Materials Used for Snow and Ice Control; New Equipment Specification; Repair, Rehabilitation and Retrofitting of Existing Equipment; Preventive Maintenance Practices; Training and Facility Management. This manual would be applicable to the winter maintenance community and could aid agencies in the prevention of metallic corrosion and the extension of service life and reliability/readiness of equipment assets.

INTRODUCTION

In cold-climate regions such as the northern United States and Canada, winter maintenance is often the activity of highest priority for transportation agencies. Large amounts of solid and liquid chemicals (known as deicers) are applied onto winter roadways to keep them clear of ice and snow. The deicers used by highway agencies are mainly chloride-based salts (Fay and Shi 2011). These chloride-based salts are a major risk to metals in equipment/vehicles, bridges and other infrastructure (Shi et al. 2013; Li et al. 2013). Structural, hydraulic, and electrical components on the maintenance equipment are vulnerable to the detrimental effects of chloride roadway deicers and their premature deterioration can negatively affect the value, performance, reliability, and service life of the equipment fleet and increase its life cycle cost and safety risk (Jungwirth et al. 2014). It should be possible to reduce the current cost of corrosion risk related to deicer exposure, if the agency can increase its investment in equipment corrosion control (Shi et al. 2015).

This article presents ideas and innovations that can be readily implemented. Its target audience include researchers and line managers responsible for fleet maintenance and/or snow and ice control operations. This work provides a comprehensive summary of how and why corrosion occurs, the effects, the costs, and practical, feasible ways to prevent and remedy corrosion. It will assist in justifying increased funding for corrosion management of equipment fleet, better on-going operational practices such as training, selection and use of advanced products, and improved facilities for cleaning of equipment.

CORROSION BASICS

Corrosion is deterioration of material due to the reaction with its service environment (Shreir et al. 1994). Corrosion may be exacerbated by the exposure of unprotected metals and alloys to a variety of chemical compounds. For instance, carbon steel, cast iron, aluminum alloys, magnesium alloys, copper and copper alloys which are used in different components of winter maintenance vehicles can be corroded by roadway deicers (Shi et al. 2013). A variety of corrosion forms can take place in a vehicle, many of which are described as follows.

General or uniform corrosion occurs when an exposed surface area of a component deteriorates at the same rate (Shreir et al. 1994; Roberge 1999). *Localized corrosion* occurs on confined areas of a surface whereas the other parts of the surface experience a much lower amount of corrosion (Shreir et al. 1994; Ok 2006). Localized corrosion can be divided into many subclasses such as pitting corrosion, crevice corrosion, intergranular corrosion, galvanic corrosion, stress corrosion cracking, corrosion fatigue, and hydrogen embrittlement (Libowitz 1979; Totten 2001). *Pitting corrosion* is a severe form of localized corrosion in which damage in the shape of deep holes occurs (Bosich 1970). *Crevice corrosion* occurs at the interface of a metal and another surface, often where a small volume of stagnant solution is contained (Xi and Xie 2002). *Filform corrosion* is a special form of crevice corrosion which happens beneath some types of coatings and has a “wormlike” visual appearance (Rust Bullet 2010; Roberge 1999). *Intergranular corrosion* occurs at grain boundaries and often spreads along adjacent grain boundaries (Xi and Xie 2002). *Galvanic corrosion* occurs when two dissimilar metals are in electrical contact. The less resistant metal (less noble) corrodes much more than the more resistant metal (more noble) which corrodes very little or not at all (Totten 2001; Xi and Xie 2002). Damage caused by the interaction of mechanical stress and corrosion is known as *stress corrosion cracking (SCC)* (Xi and Xie 2002). *Corrosion fatigue* is a special kind of SCC caused by the combined effects of cyclic stress and corrosion (NASA-KSC 2014). Rust jacking is a kind of corrosion fatigue due to the accumulation of crevice corrosion products (Lockridge 2007). Fretting corrosion happens at the interface between contacting, loaded metallic surfaces in the presence of slight vibratory motions (NASA-KSC 2014).

CAUSES AND EFFECTS OF CORROSION

General corrosion is often due to poor material selection for a corrosive medium (Gooch 2007; NACE 2014a). Localized corrosion can lead to premature and unpredictable failures; therefore, localized corrosion can be more dangerous than general corrosion (Shi et al. 2013). Pitting is one of the most destructive forms of corrosion (Papavinasam 2013). It is often hard to distinguish pits due to their small diameter and often times they are concealed (Gadag and Ntyananda Shetty 2007). Crevice corrosion is often observed on vehicles in narrow gaps, such as the dual chassis rail sections which can force rails apart; due to rust jacking (Mills 2012). "Lacquers and "quick-dry" paints are most susceptible to filform corrosion. Their use should be avoided unless absence of an adverse effect has been proven by field experience" (Rust Bullet 2014). Intergranular corrosion is due to impurities present at the grain boundaries, enrichment of an alloying element, or depletion of alloying elements at the grain boundaries (Totten 2001). In stainless steels, depletion of chromium in the grain boundaries leads to intergranular corrosion (Totten 2001). It often happens in heat affected zones a short distance from the weld (NASA-KSC 2014). Aluminum alloys are susceptible to galvanic corrosion when coupled to steel (Levelton Consultants 2007). A stainless screw used with cadmium plated steel washer created galvanic corrosion (Rust Bullet 2014). SCC can lead to serious outcomes because it can occur at stresses inside the range of nominal design stress (Committee on Conservation of Materials 1977; NPL 2000). When a metal is continually exposed to a corrosive media, corrosion fatigue may happen at even lower loads and in a shorter time frame than anticipated (NACE 2014b). Rust jacking has been observed when corrosion develops between the brake shoe table and the brake lining (Lockridge 2007). Fretting corrosion is usually found in machinery, bolted assemblies, ball or roller bearings, and between connectors and terminals in electrical components (Rust Bullet 2010; GM Techlink 2009).

In a survey conducted by Shi et al. (2013), survey respondents were asked to rank the risk of metallic corrosion to the types of equipment the respondents' agency owns. Dump trucks were listed as greatest concern of having "very high" risk of metallic corrosion (49%), followed by liquid deicer application equipment (34%). Survey respondents were also asked to rank the risk of chloride deicers to various components of their agency's vehicles/equipment. Electrical wiring was identified as highest risk to suffer from deicer corrosion (4.5), followed by Frames (4.0), Brackets and supports (3.9), Brake air cans (3.9), Fittings (3.9), Spreader chute (3.9), etc. Liquid storage tanks have the lowest perceived risk to deicer corrosion (2.8).

Figure 1 shows various parts of winter maintenance vehicles corroded by deicers. This figure includes (a) double rail truck in which corrosion between the rails forced them apart (cracked frame rail); (b) brake drum corrosion; (c) 2007 MACK rotted latch (made of carbon steel). For prevention, addition of Polly tandem fender guards to all newly specked plow trucks has been recommended; (d) frame rail scaling, has led to chassis destruction. The prevention method is using E-coating reinforced

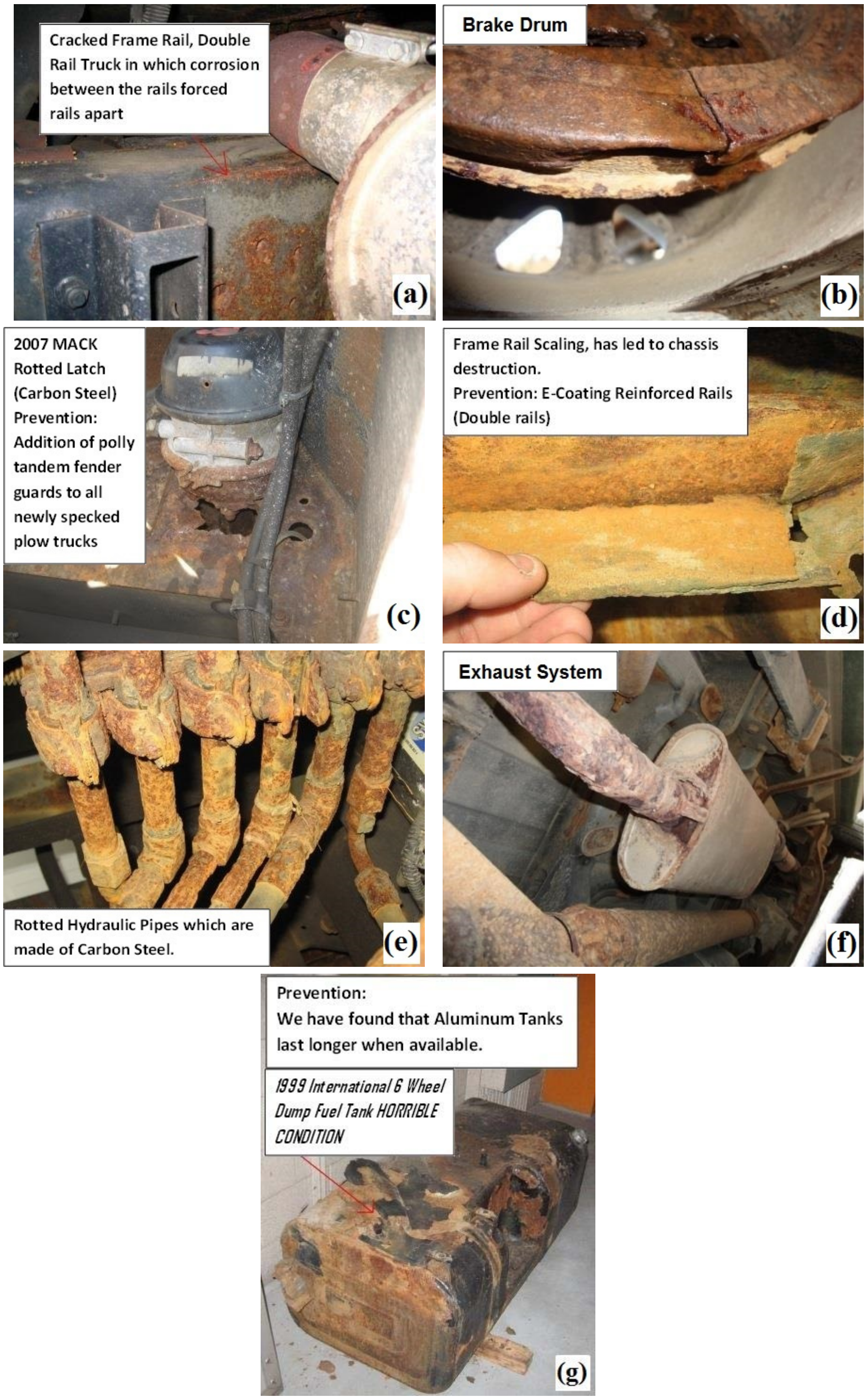

Figure 1. Images of deicer corrosion damages on (a) frame, (b) bake drum, (c) latch, (d) chassis, (e) hydraulic pipes, (f) exhaust system, and (g) fuel tank (Shi 2014; courtesy of Rhode Island Department of Transportation).

rails (double rails); (e) rotted hydraulic pipes made of carbon steel which can be corrected by using stainless steel hydraulic pipes; (f) exhaust system corrosion; (g) 1999 International 6 wheel dump fuel tank which has horrible condition due to corrosion. Replacing carbon steel tank by aluminum tank is the prevention method (Shi 2014).

In a national survey, Shi et al. (2013) sought feedback on the types of deicers that vehicles and equipment were exposed to. The following four deicers; salt, pre-wetted salt, sand/salt blend and sodium chloride brine, were listed as "frequently or very frequently exposed" to vehicles/equipment in this survey. It is interesting to note that only 12.6% of the survey respondents used non-chloride deicers. The survey respondents also indicated that in areas with cold-temperature, the use of corrosion-inhibited magnesium chloride (vs. the non-inhibited chloride brines) to pre-wet sand could lead to reduced corrosion risk to vehicles and equipment (Shi et al. 2013).

The survey by Shi et al. (2013) also identified annual expenditures for managing deicer-related metallic corrosion by agencies that report it as being a significant issue. The average estimated annual costs per agency for corrosion management in six areas are as follows:

- Training programs ($190,938);
- Materials selection ($320,667);
- Design improvements ($45,000);
- Corrosion monitoring and testing ($10,000);
- Proactive maintenance ($171,424);
- Reactive maintenance ($325,000).

MATERIALS USED FOR SNOW AND ICE CONTROL

Generally, the materials used for snow and ice control are either chemically inert or active. Inert materials have no de-icing properties; they simply provide temporary traction on icy pavement. Sand, finely crushed "rock dust," pumice, cinders and slag from coal-fired power plants or foundries, sawdust, even crushed oyster shells in coastal areas have been used.

Chemically active de-icing materials are:

- Sodium chloride – either as solid rock salt or in solution as brine; not effective below 15°F. Sodium chloride is corrosive especially on steel components, under immersion and in arid environments (Mills 2012);
- Calcium chloride – either as solid or liquid. It works at colder temperatures than sodium chloride (down to -20°F). Calcium Chloride is very corrosive especially for aluminum parts (Xi and Xie 2002);
- Magnesium Chloride – either as solid or liquid; works at colder temperatures than sodium chlorides (down to 5°F). It affects aluminum, and is more corrosive than sodium chloride under humid conditions (Mills 2012);
- Potassium Acetate;

- Calcium Magnesium Acetate;
- Sodium Acetate;
- Potassium Formate (Fay and Shi 2012).

In the last two decades, acetate-based deicers such as potassium acetate (KAc), sodium acetate (NaAc) and calcium magnesium acetate (CMA) have gradually replaced urea as a freezing-point depressant and are used by some transportation agencies to treat winter roadways. In addition, formates such as sodium formate (NaFm) and potassium formate (KFm), as well as bio-based products have emerged as potential alternative deicers (Fay and Shi 2012).

NEW EQUIPMENT SPECIFICATION

Materials selection. In past decades, countless amounts of time and effort have been dedicated to anticorrosion research. Advancements in technology have allowed for various improvements in material selection to minimize corrosion. In general, magnesium alloys and mill product forms of aluminum alloys 2020, 7079, and 7178 should not be used for structural applications. The use of 7xxx-T6 Al alloys should be limited to a thickness of no more than 0.080 inches or 2.0 mm (DOD 2007). Where stress corrosion cracking is the main problem, 7075-T6 can be replaced by 7050-T7451. Moreover, by this strategy the manufacturing time will be shortened (Phan 2003). Corrosion resistance of aluminum alloys is optimum in environments with a pH range of 4 to 9. Out of this range the aluminum oxide film that forms on the metal surface and protects the metal degrades (Reboul and Baroux 2011). Higher carbon content and hardness in steel would make it susceptible to SCC or embrittlement (Light Truck 1999). In SCC of austenitic stainless steel by chlorides, substitution of duplex stainless steels will often eliminate the problem (NACE MR0175/ISO15156). Using steels containing molybdenum such as stainless steel 316 can reduce pitting corrosion (Shi et al. 2013). Intergranular corrosion can be reduced by using the stabilized (321 or 347) or low-carbon (304L or 3I6L) stainless steels (NASA-KSC 2014).

Design improvements. If an operating environment is considered to be corrosive, engineered design of components should reflect this situation. For example, a design should be used to avoid creating locations where water may accumulate or, if that is not possible, include holes for drainage (Agarwala 2004; Shi et al. 2013). Minimum diameter for all drains should be 3/8 inches or 9.5 mm (DOD 2007). The drainage holes should be shielded or oriented to avoid direct road splash (Roberge 1999). Avoid crevices and design features that make it difficult for protective coatings to function (e.g., sharp corners) (DOD 2007). Edges and corners are difficult to coat uniformly, and thinly coated protrusions are susceptible to corrosion. On the other hand, the simple cylindrical structural members are preferred because they allow for ease and uniformity of paint application as well as convenient inspection (Aluminum Association 2001). Stress corrosion cracking can be prevented by removing notches and other stress-concentrating features. Rounded filets and angles also reduce stress concentrations (Shi et al. 2013). Lowering the cyclic stresses and using corrosion control are the best ways to eliminate the corrosion fatigue (NASA-KSC 2014). The most promising approach for prevention of rust-jacking is applying an adhesive membrane between a new shoe table and the lining material. This heat-activated material melts under the application of the brakes, sealing the lining to the brake shoe. Once the adhesive membrane has set up, that brake shoe cannot be re-lined (Lockridge 2007). The maximum allowable size for a crack in brake lining is 1/16-inches (1.59 mm) wide and 1.5-inches (38.1 mm) long (Winsor 2014). Crevice corrosion can be minimized by proper design of welded joints and gaskets that minimize crevices, and also sealing the crevices and periodic cleaning. Contact between dissimilar metals can lead to galvanic corrosion and should be avoided. Where it is not possible, both metals should be coated (DOD 2007). Filform corrosion can be avoided by using a coating with low water vapor transmission characteristics and excellent adhesion. Zinc-rich coatings can be considered for carbon steel because of their cathodic protection ability (Rust Bullet 2014). The best way for reducing fretting corrosion is cleaning the affected surfaces and applying appropriate lubrication (GM Techlink 2014).

Following considerations have been introduced by Resene (1999) for welded joints:

- Eliminate the weld splatter using blasting or chipping.
- Rough welding should be ground smooth.
- If feasible, welds should be double coated.
- Where corrosion is possible, use continuous welds instead of discontinuous welds (tack or skip welds).
- Remove brackets and extra metal followed by ground smoothing areas of previous contact.
- Remove weld flux after finishing of welding.

REPAIR, REHABILITATION, AND RETROFITTING OF EXISTING EQUIPMENT

Municipal Public Works and State Departments of Transportation (DOTs) have long dealt with the problem of keeping worn equipment in service without expending exorbitant amounts of money, effort, and time. Maintenance and operation managers must decide what is not economically feasible to save; what can be retained but needs extensive work and lastly, what still has a long service life but needs some minor work to keep in top condition (Honarvar Nazari et al. 2014). The evaluation process for fleet includes:

- A thorough inspection and operational check of each unit;
- An initial itemized list of repair/maintenance work for each vehicle;
- A general assessment rating using a standardized grading approach;
- A priority ranking for each item, for example: Critical, Urgent, Needed, Recommended;
- A detailed cost estimate for each vehicle;
- A determination of expected service life if repairs/rehab is done;
- A decision for the course of action for each unit of equipment (Routine Maintenance Only; Selective Repairs; Full Repair/Rehab; Defer and Re-

evaluate within __ days; No Maintenance or Repair-Deadline; Remove from Fleet by Sale, Trade-In, Transfer); and

- Final cost estimate based on evaluation (Honarvar Nazari et al. 2014).

Restoration differs from repair mainly based on the severity and extent of remedial work needed to return the vehicle to full operational capability and reliability. For example, removing scattered rust spots and repainting a dump body is a repair; replacing the entire body and material applications systems is a restoration. Restoration usually requires a considerable expenditure of time and money and is typically justified to keep a unit past its expected service life. An agency may be forced into such a situation if funds are not available to purchase a new similar vehicle or if the unit will be placed into a reserve status. Snow and ice operations require a certain number of specially equipped vehicles; a type that is not easily obtainable by renting or leasing on short notices (Honarvar Nazari et al. 2014).

PREVENTIVE MAINTENANCE PRACTICES FOR EQUIPMENT ASSETS

Two primary ways to deal with equipment corrosion are reactive and preventive. Reactive methods are used to deal with existing corrosion by cleaning corroded parts with a rust removing compound, or replacing the ones that are too far gone for rehabilitation. A reactive treatment may in some cases be the most cost effective means of dealing with corrosion if the parts are easy to clean, or easily replaced and fairly inexpensive. Preventive methods are the proactive strategy which may involve the use of corrosion resistant materials for equipment components, dielectric grease, enclosed wiring connections, the use of sacrificial anodes, the use of coatings, and the use of corrosion inhibited products. Frequent and regular washing of equipment can be considered a preventive strategy (Mills 2012). Washing is a very effective way to reduce equipment corrosion (Nixon and Xiong 2009).

Here are some points that should be considered in a washing program:

- Wash vehicle daily especially following anti-icing activity to reduce chloride salt build up (Mills 2011).
- Washing should be concentrated on trouble spots like frame rails, brake components, underneath of the chassis and other areas that tend to collect materials (Shi et al. 2013; Winsor 2014).
- Washing should be followed by localized cleaning and then fast drying (Shi et al. 2013).
- Perform routine washing preferably with hot water (AFCPCO 2003).
- Preferably do not use a pressure washer, because water can be forced into areas and cannot escape which leads to corrosion (Mills 2011).
- Use low pressure wash and high volume (flow rate) of about 300 psi/300 gpm (Clonch 2014).
- Use physical action together with washing to remove the road salt (Lockridge 2007).
- Washing with water alone or soap and water together is not sufficient for removing residual salt. The use of salt neutralizers (removers) is strongly recommended (Monty et al. 2014).

- Use salt neutralizers after pressure wash as high pressure wash can cause capturing of salt and water at crevices which can cause crevice corrosion (Monty et al. 2014).
- The effectiveness of salt neutralizers is alloy specific; therefore using the incorrect salt neutralizer can even accelerate corrosion (Monty et al. 2014).
- More cleaning liquid is not necessarily better - in some cases, a high concentration of washing compound may actually attack some of the plastics that are there to provide corrosion resistance (Lockridge 2007).
- Once active corrosion of metals has started, washing should be coupled with other means, e.g., applying spray-on corrosion inhibitor (Shi et al. 2013; Li et al. 2014).
- Wherever possible, agencies should consider de-humidified storage of their equipment and vehicles, or at least store equipment in dry climate after washing (Shi et al. 2013).

Based on extensive research, the following proactive methods for reducing corrosion in different parts of the winter maintenance vehicles are provided.

Electrical issues. It is recommend to eliminate the junction boxes wherever possible, and relocate them to inside the cab off the floor. Further recommendations are provided as follows (Lockridge 2007; Shi et al. 2013; Mills 2011 and 2012; WSDOT 2011; TRB 1991; DoD 2014; Winsor 2014; Smith 2014; Chupas 2014).

- Move the relay and circuit breaker for spreader controls which are mounted in the battery box into an enclosed box inside the cab of the truck;
- Move the electronic control module (electronic brain) inside cab;
- Mount electrical junction blocks in cab;
- Position wiring to reduce damage to the outside casing of wires;
- Use weather tight electrical connections;
- Install sealed LED lighting; keep water drained from the air conditioner system;
- Do not probe the wires to test for continuity and avoid any damage of wiring insulation;
- Avoid splicing into wiring and do not pierce wire jacketing;
- Use a heat shrink terminal that seals out moisture especially in the case of any wiring repair;
- Use high-quality weather-proof terminations (e.g., buss-style connectors) and compression fittings in addition to shrink wrapping susceptible electrical wiring components;
- Use dielectric silicone for sealing damaged areas or connections;
- Install sealed wiring harness front to rear;
- Seal electrical systems (e.g., connectors, switches, and circuits) by using adhesives, deadeners, and sealers (e.g. polysulfide);
- Use sealed connection wires rather than butt connections;
- Wire probes should be avoided to prevent intrusion;
- Apply a non-conductive, non-sodium based di-electric grease on all electrical connections (e.g. plugs, sockets, pigtails, battery terminals, etc.);

- Make clean the electrical connectors on a regular basis (at least every six month) with water (not soap) and a wire brush, and re-grease with dielectric grease;
- Minimize connectors to the extent possible by using continuous wiring;
- Use anti-corrosive spray for protecting the battery posts and terminals;
- Do not apply paint to the rubber seals around lights;
- Install modified protective cover for battery.

Brake components/chassis. Maintenance crew should inspect all brake components even by removing brake drums for checking the entire lining surface and the brake shoe web, rollers, cam, etc. Additional recommendations are provided as follows (Lockridge 2007; Shi et al. 2013; Mills 2011 and 2012; WSDOT 2011; TRB 1991; DoD 2014; Winsor 2014; Smith 2014; Chupas 2014).

- Install corrosion sealed air brake chambers;
- Spray on protective coatings on all brake valves; for all vehicles, especially five years and older pull brake drums on a regular basis;
- In the rebuilding process specify rust-proof painted and epoxy-coated brake shoes;
- Use sealed brake canisters and sealed protective boxes surrounding hydraulic components, and replace with new style when necessary;
- Require throttle, brake, and clutch pedals to be suspended in specifications (vs. floor mounts);
- Specify self-healing undercoats, full fenders and fender liners for chassis;
- Use rubberized undercoating for aluminum brake valves;
- Ensure automatic slack adjusters (ASAs) are thoroughly lubed and there is no evidence of internal rusting;
- Install a large full width, full height under chassis sand guard on all front discharge sanding bodies;
- Use plastic quick release brake valves instead of aluminum.

Frame/body/beds and other parts. For the better maintenance of frame/body/beds and other parts, agencies should perform the following recommendations (Lockridge 2007; Shi et al. 2013; Mills 2011 and 2012; WSDOT 2011; TRB 1991; DoD 2014; Winsor 2014; Smith 2014; Chupas 2014).

- Replace certain corrosion-prone components (e.g., carbon steel) with corrosion-resistant materials such as stainless steel, aluminum alloys, and plastics-and coated metals, such as clad steel and galvanized steel or non-metallic (e.g. poly or composite materials) wherever possible;
- Inspect and replace corrosion-prone components on a regular basis;
- Steel fuel, hydraulic and air tanks can be replaced with aluminum tanks;
- Replace standard E-coat steel painted wheels with powder coated versions;
- Use powder coating for fuel tank and frame rails;
- Replace carbon steel oil pans with more expensive zinc coated or stainless steel oil pans;
- Consider buying stainless steel truck boxes, pre-wetting tanks, and sanders;
- Use zinc anodes in solution tanks; use stainless steel couplers;

- Use stainless steel under tailgate spreaders; use poly-faced snowplows to reduce corrosion and also lessen weight;
- Use stainless steel cooler lines for Allison transmissions;
- Use greaseable tailgate linkages and attach them to on board automatic lube system;
- Use high-quality primers and topcoats in equipment specifications;
- Do not use wheels with cracks, dents, leaks, severe wear, or rust pitting;
- Truck frames should be coated, sandblasted and painted as needed;
- Paint thickness on each side of wheel mounting face should not be more than 3 mils;
- Use rear mounted material application equipment as much as possible;
- Install zinc nickel alloy engine oil pan;
- Install grit guards on wheels (prevents wheels from rusting together and to the axle hub);
- Wrap hydraulic fittings with anticorrosive wrap;
- Seal frame rail split;
- Use glad hand seals with dust flaps for air system.

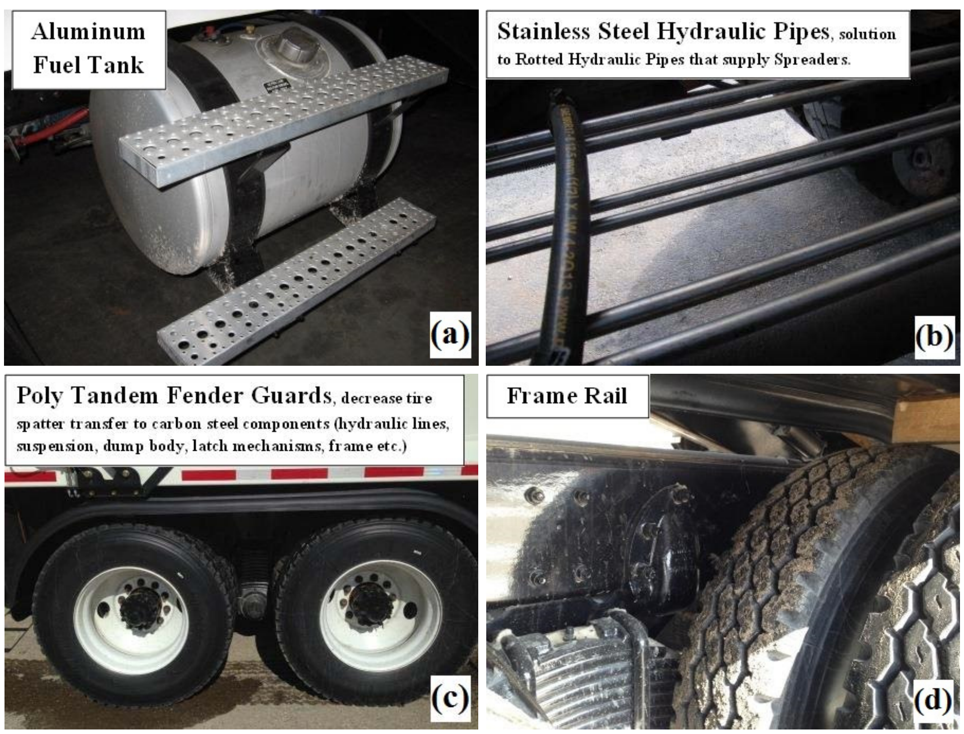

Figure 2. In-field success of some proactive maintenance methods (a) aluminum fuel tank, (b) stainless steel hydraulic pipes, (c) poly tandem fender guards, (d) E-coated frame rail (courtesy of Rhode Island DOT).

Figure 2 shows in-field success of some proactive maintenance methods applied by Rhode Island DOT such as (a) aluminum fuel tank instead of carbon steel fuel tank; (b) stainless steel hydraulic pipes, solution to rotted hydraulic pipes that supply spreaders; (c) poly tandem fender guards which decrease tire spatter transfer to carbon steel components (e.g. hydraulic lines, suspension, dump body, latch mechanisms, frames, etc.); (d) E-coated frame rail in good condition (courtesy of Rhode Island DOT).

TRAINING AND FACILITY MANAGEMENT

Operators, including contractors, need to know the basic characteristics of each product being used, including inert abrasives. These factors include: effective temperature range, corrosiveness, toxicity, environmental impacts, recommended application rates, and special handling precautions. For example, mixing calcium chloride with some organic-based anti-icers may result in clogged sprayers. Or, that a lot of salt is no more effective in certain conditions than lesser amounts. *Mechanics* who repair and maintain the fleet need basic information on the materials used and the associated characteristics, especially an understanding of the corrosiveness of each chemical. This will help mechanics identify potential locations on the vehicles susceptible to corrosion and what preventive measures can be taken. *Supervisors and managers* are responsible for making sure that operators and mechanics and others involved in transporting, handling, and storing materials have the proper training. It is also their responsibility to ensure compliance with the established policies and practices both at the facilities and in the field. Lastly, supervisors and managers need to be able to cogently explain not only to staff but to upper jurisdictional leadership, the media and the public why, how, and when materials are used (Honarvar Nazari et al. 2014).

CONCLUDING REMARKS

- Existing knowledge about the anti-corrosion performance of various materials and design configurations in various deicer-laden service environments should be utilized to refine the equipment purchasing specifications developed by the transportation agencies.
- Maintenance and operation managers must decide what is not economically feasible to save; what can be retained but needs extensive work and lastly, what still has a long service life but needs some minor work to keep in top condition.
- Agencies should implement an extensive preventive maintenance program that may involve use of salt removers (also known as salt neutralizers) together with routine washing to keep the corrosive salt from building up; protecting of electrical components by sealing or moving them to inside the cab; reapplication of post-assembly coatings; spray-on corrosion inhibitors and many other operational changes to improve the service life of DOT vehicles. This can be supplemented by corrective maintenance practices to minimize the negative impact of deicer corrosion to equipment asset.

- Supervisors are responsible for ensuring compliance with procedures and practices regarding vehicle inspection and operation, and also training of staff and contractors.

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Introduction of a Chemical Grouting Method for the Crack Repairing of Asphalt Pavements

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Abstract

Crack repairing is considered a cost-effective technique in preventive maintenance, provided that right repairing method and material are selected and properly applied. By means of timely and effective crack repairing manners, the service life of pavement could be increased and leads to significant cost savings for the highway management system. In this study, a chemical grouting method that usually used in the repair of cement concrete structures was employed and modified for the crack repairing of asphalt concrete pavements. In order to evaluate the feasibility and effectiveness of the proposed method, in-situ experiments were performed on several typical pavement sections with all kinds of cracks. Through long-term observation and by comparison with the conventional crack repairing method, the chemical grouting method was proved effective for the crack repairing of asphalt pavements, and exhibited desirable durability under heavy traffic loads.

INTRODUCTION

Cracking is one of the annoying distresses in asphalt concrete pavements, which not only affects the serviceability of pavement, but deteriorates the performance of the whole pavement structure. When a crack develops on the surface of pavement, it is a sign of reduction in pavement integrity, and under the combined actions of traffic loads and environmental media, failure to repair cracks may lead to accelerated deteriorations of the pavement in the forms of crack growth, raveling, secondary cracks, and even subgrade damages.

With the increasing traffic volumes, larger proportion of the annual transportation budget is spent on pavement maintenance, particularly in the countries having large networks of roads and highways, and crack repairing has become one of the most common and important pavement maintenance activities worldwide (Lugmayr et al. 2009; Brakey 2000). Although cracks in pavement are inevitable, repairing efforts to cracks could still improve the performances of the pavement and extend its service life (Shuler 2007). The effect of crack repairing is mainly determined by the properties of repairing materials and the related repairing methods. Appropriate repairing method and material can not only guarantee the repairing effect but increase the cost-effectiveness of preventative maintenance (Ponniah and Kennepohl 1996; Smith and Romine 1999). It was reported that effective and timing repairing for cracks could extend the service life of a pavement up to 5 years and save the long-term maintenance costs for 45-50% (Ponniah and Kennepohl 1996; Freeman and Johnson 1999).

Many crack repairing methods have been developed for asphalt concrete pavements, which include crack sealing, crack filling, sealcoats, and overlays (Springfield 2009). The crack sealing and filling method with asphaltic sealant (as shown in Figure 1) are currently the most commonly used method in preventative maintenance services, which could maintain a smooth riding surface, seal the open crack, and thus extending the useful life of the pavement to some extent (Yildirim et al. 2002; Yildirim et al. 2006).

a) b)

Figure 1. Conventional crack repairing method.

The conventional crack repairing method usually features high efficiency and cost-effectiveness due to its simple and easy practice by using special equipments, and the excellent compatibility between asphaltic sealants and asphalt concrete substrates endows it with some inherent advantages. However, this method can only seal or partially filled the open crack to prevent water and debris from entering the underlying soils, and it has no benefits on restoring the bearing capacity and continuity of load transfer for the pavement. Moreover, the high temperature sensibility and low abrasion resistance of the asphaltic sealants could significantly affect the durability of the repair. In many cases, re-opening in a short-term is prevalent for the crack repaired with the conventional repairing method, especially for the pavement under severe weather conditions or heavy traffics.

OBJECTIVE AND SCOPE

A chemical grouting repairing method usually used for the repair of cement concrete structures was introduced in the study. In order to achieve desirable repairing effect for asphalt concrete pavements, a special chemical repairing material was developed compatible with this method.

In-situ experiments were carried out to validate and improve the suitability of the proposed method. In addition, the repairing effect of the proposed method was also evaluated by comparison with the widely-used conventional crack repairing method.

MATERIALS

The chemical repairing material used for the study, named EMP, was a polymer-based composite synthesized by three widely used polymers: epoxy acrylate (EA), methyl methacrylate (MMA), and polyurethane (PU) prepolymer. With respect to those components, EA is the production that synthesized by the esterification reaction of open-looped epoxy resin and unsaturated monobasic acid (e.g. acrylic acid), which combines the excellent adhesive properties of epoxy resin, as well as the processability of unsaturated polyester; MMA is an active diluents used to adjust the viscosity of the pure EA to improve its flowability, operability and groutability of the mixture; while PU is a resilient, flexible and durable manufactured material that can improve the ductility and flexibility of the repairing material, and thus enhance the compatibility between the repairing material and asphalt concrete substrates. In addition to those polymers, evocating, accelerating and cross-linking agents are required to motivate the interpenetrating reactions of the mixtures to form stable solidification structures.

The compositions and proportions of the EMP are presented in Table 1. Assuming the content of EA is 100%, the contents of the other components in the mixture are all calculated based on that.

Table 1. Compositions and proportions of the repairing material.

Component	*Chemical name*	*Content*	*Proportion* [a] *(ratio by weight)*
EA	epoxy acrylate	100%	
MMA	methyl methacrylate	50%	MMA:EA=0.5
PU prepolymer	polyurethane prepolymer	40%	PU:EA=0.3
Evocating agent	benzoyl peroxide dimethyl phthalate	1.0% to 3.0%	1.0% to 3.0% of EA
Accelerating agent	N,N-dimethylaniline styrene solution	1.0% to 3.0%	1.0% to 3.0% of EA
Cross-linking agent	1,4-butanediol dehydration	0.8%	2.0% of PU
Coloring agent [b]	hydrocarbon black	2.0% to 3.0%	2.0% to 3.0% of EA

NOTES: [a] The proportions of evocating and accelerating agents are dependent on the content of EA, while the proportion of cross-linking agent is dependent on the content of PU. The proportions of those agents also vary with the operation temperature. [b] Coloring agent is not an essential component for the material.

Physical and mechanical properties of the EMP were evaluated through a series of experiments in the laboratory, such as viscosity test, volume shrinkage test, compression and tension tests, and flexural strength test. The primary properties of the EMP used for the study are summarized in Table 2. For the convenience of construction, the material was modulated with different viscosity and hardening time for sealing and grouting purposes.

Table 2. Properties of the repairing material.

Property	*Test result*
Viscosity [a] (mPa·s)	used for sealing: 827 used for grouting: 315
Hardening time (min.)	used for sealing: 15.6 used for grouting: 33.7
Tensile strength (MPa)	12.0
Compressive strength (MPa)	28.3
Flexural bending strength (MPa)	8.64
Elastic modulus (GPa)	1.32
Elongation at break (%)	12.1
Volume shrinking ratio (%)	1.98

[a] The viscosity of the material can be adjusted by the amount of diluents used in the mixture, and the hardening time of the material can be adjusted by the quantities of evocating and accelerating agents.

Based on trial experiments in the field, the repairing effect was affected by the viscosity of the repairing material. Too large viscosity of the material would decrease its processability and operability, while too low viscosity would cause difficulties to control the infiltration of the material during repair. In order to achieve desirable repairing effect, the optimal viscosity of the repairing material needs to be controlled within the range from 600 to 1500 mPa·s for sealing and 200 to 600 mPa·s for grouting.

METHODOLOGY

Crack grouting repairing is a method using grooving, drilling, and grouting efforts to assist the chemical repairing material to permeate into the crack networks and bond the damaged surfaces. Due to the excellent flowability and adhesive properties of the chemical repairing material, the method could not only seal the open cracks but mend the deep damages to achieve desirable durability and even partially restore the structural function of the pavement structure. The method contains the main procedures as following.

Grooving. Prior to repairing, a groove needs to be cut along the targeted crack using a router, as shown in Figure 2. The groove can cut off the damaged edges of the crack and provide clean and firm surfaces for the repair, which can improve the bonding strength between the repairing material and asphalt substrate. In addition, a reservoir for repairing material created by the groove wider than the original crack can also improve the resistance of the repairing material to the stresses induced by traffic and temperature.

a) b)

Figure 2. Grooving for cracks.

Blowing and vacuuming. After grooving, the crack needs to be cleaned and dried with blower or vacuum to guarantee the adhesion between repairing material and asphalt concrete substrates. Due to the characteristics of the EMP, slightly damp but no trapped water in the crack is an acceptable condition for the repairing.

Drilling. For placing the grouting needles, small holes need to be drilled along the crack. The spacing between drilling holes are an important factor for the effect of grouting, normally the spacing is controlled within 30 to 50 cm depending on the general conditions of the crack. The location of the drilling holes should be chosen on the intersection points of crack networks and the region with severe edge deteriorations. For the area with extremely intensive cracks, the spacing could be reduced eligibly.

Installation of grouting needles. After drilling, grouting needles are installed in the drilling holes, as presented in Figure 3. The grouting needle consists of a threaded rod, two rubber bushings and a needle beak with inverted valve. The diameters of the needle used in the study were 10mm and 15 mm, the length of the needle is chosen depending on the type, depth and severity of the targeted crack.

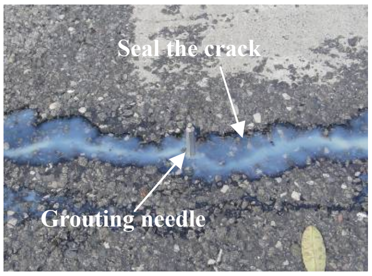
During installation, the needle needs to be tightened with a T wrench, and the rubber bushings should be compressed and bulged in the drilling hole so that the needle would not be pushed out of the hole under grouting pressures.

a)

b)

Figure 3. Installation of grouting needles.

Sealing. Prior to the grouting process, the groove need to be sealed to ensure that during grouting the repairing material would infiltrate into the deep cracks under pressure. In order to guarantee the sealing effect, the chemical mixture with relatively high viscosity (600 to 1500 mPa·s) was used for sealing. The mixture with high viscosity could adhere to and stick on the crack openings and thus seal the upper part of the crack.



a)

b)

Figure 4. Crack sealed with chemical repairing material.

Grouting. After the sealing material is solidified, grouting can be processed with a grouting machine, as illustrated in Figure 5. Pressure grouting is the most important and skillful operation that requires special attentions: 1) In the beginning of grouting, the needle may be slightly pushed up by the grouting pressure, but it would be self-stabilized with the repairing material infiltrates into the open cracks and pores under the pressure; 2) Due to the air trapped in the crack, the pressure of grouting needs to be controlled less than 0.2 MPa to exhaust the air, and then increasing the pressure gradually to a stable status. Commonly, the grouting pressure is controlled within 0.3 to 0.5 MPa; 3) During grouting, frequent-but-small injection of repairing material in each grouting point could achieve better filling effects than the injection only in one operation.

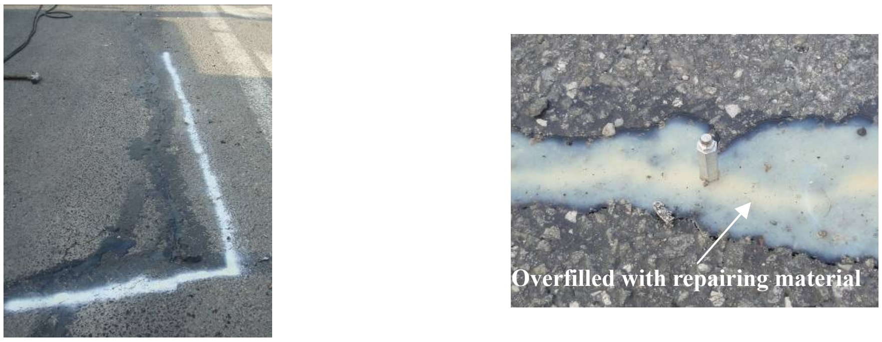
In order to obtain a steady grouting, injection with uniform speed is required, and in some occasions, the methods such as intermittent injection, jump-hole grouting and extended grouting interval can be used to avoid grouting failure.

a)

b)

Figure 5. Grouting for the repair.

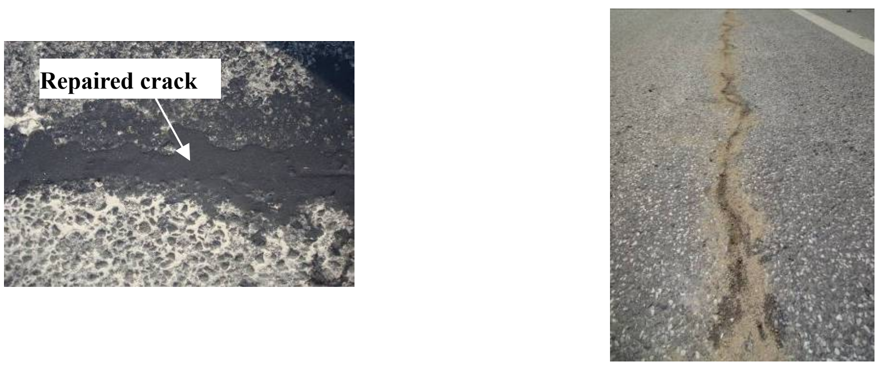
Re-grouting and refilling. Before the repairing material used for grouting is solidified, re-grouting the cracks in the repaired pavement section to ensure that each grouting point is fully filled. In addition, the slotted groove along the crack also needs to be refilled by the repairing material due to the loss and sinkage caused by volume shrinkage and infiltration.



a) b)

Figure 6. Re-grouting and refilling for the repair.

Curing. After re-grouting and refilling, absorbing materials are spread on the crack to cover the unsolidified sealing material (shown in Figure 7). Clean sand, fly ash and stone dust are the most commonly used absorbing materials in the curing process. Those materials sprinkled evenly on the repaired crack can form a hard-shell overlying layer in the groove to increase the skid resistance of the repaired surface, and meanwhile improving the abrasion resistance of the repairing material. When the repairing material is completely solidified, the redundant absorbing materials on the pavement surface could be swept away to restore the traffic.



a) b)

Figure 7. Curing for the repaired crack.

REPAIRING EVALUATION

The pavement for field experiments was located in a subtropical area with humid and rainy climate. The average temperatures in summer and winter seasons of the application sites are about 28℃ and 11℃, respectively, and its annual rainfall is more than 1500 mm. The designed speed of the selected pavement sections is 100 k/h, and their average daily traffic is around 90-thousand vpd. Long-term observation was performed on the repaired cracks in the typical pavement sections. Figure 8 shows the status of cracks after being repaired for certain periods of time.

According to the results, there was no apparent abrasion wear and damage observed on the appearance of most cracks in 6 months. With the service time increased to 12 months, the superfluous repairing material close to the crack was worn out, but the overband caps on the top of the repaired cracks were still well-preserved. While, after being repaired for 18 months, slight damages and spalling have emerged on the edge of overband for some cracks, because those thin overlying were too vulnerable to resist repeated heavy traffic loads, but no adhesion failure was observed in the material-asphalt interface zone of the repaired cracks.

a) 6 months b) 12 months c) 18 months

Figure 8. Cracks after repaired for certain periods of time.

Based on the observation results, most of the asphaltic sealants used to repair the cracks in the conventional method were worn off after about 12 months in service, especially for the large cracks with subgrade damage.

CONCLUSION

A chemical grouting method was employed in the study for the crack repairing of asphalt concrete pavements. Based on the study, the following conclusions can be drawn.

1) With the proper repairing material, the proposed crack repairing method was proved feasible and effective for the crack repairing of asphalt concrete pavements.

2) According to the results from long-term observation, the repair made by the chemical grouting method showed better durability than the repair made by the conventional method, especially for the repair of structural crack and the crack with subgrade damages.

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An Experimental Study on the Repair of Deteriorated Concrete by the Electrochemical Deposition Method

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Abstract

Cracks significantly deteriorate the in-situ performance of concrete members and structures. Electrochemical deposition tests were performed to repair different types of damaged concrete specimens, such as specimen with the load-induced cracks and with the internal defects. Based on the authors' previous work, electrochemical deposition tests are performed with the porous concrete specimens to assess its healing effects. Three kinds of experiments have been utilized to evaluate the deposition healing effectiveness. The experimental results show that the flexural strength improves after healing. The average ultrasonic pulse velocities increase from 3,546m/s before healing, to 3,617 m/s after 14-day healing, to 3,656m/s after 35-day healing, meanwhile the average porosities reduce from 0.2999, to 0.26245, and finally to 0.24092.

INTRODUCTION

Cracks typically occur in concrete members and structures, like the marine structures and underground structures, due to the influences of the surrounding environment, such as the construction disturbance, underground water and train vibrations. Cracks may seriously deteriorate concrete's strength, performance and durability (Mehta, 1997), particularly in cold climates area where the ice melting products are widely used. Electrochemical deposition method (EDM) is a new technique for repairing concrete cracks in an aqueous environment. It has been developed in the past 20 years and applied to marine structures or other situations in which the traditional repairing methods are inadequate (Yokoda and Fukute, 1992; Sasaki and Yokoda, 1992).

After the method was used to repair the marine structures, many experimental studies are performed trying to apply EDM to repair the land concrete structures (Ryu, 2003a; Monteiro and Ryu, 2004; Ryou and Otsuki, 2005; Mohankumar, 2005). The set-up of the EDM tests for the land concrete structures can be summarized as follows: The embedded rebar of the concrete structure and the titanium mesh are connected to the negative terminal (-) and positive terminal (+) of the power supply, respectively, as displayed in Figure 1. Both the concrete structure and titanium mesh are immersed in the aqueous solution. There are continuous electrochemical deposition products in the concrete member or structure when an appropriate direct current (DC) is maintained in the system. The deposition products can repair the defects (like the micro-pores or cracks). Meanwhile the harmful ions will be removed from the specimen, which will reduce the damage of corrosion (Ryu and Otsuki, 2002; Zhu et al., 2014; Yan et al., 2013).

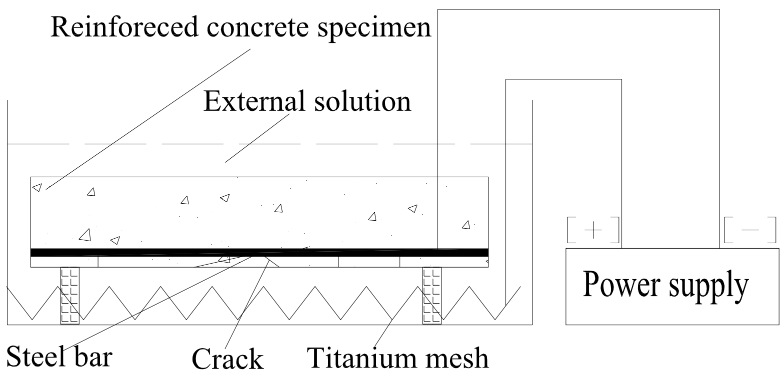

Figure 1. The schematic plot of the EDM.

To have a better understanding of this new and promising healing method for concrete structure in an aqueous environment, many notable studies (Ryu and Otsuki ,2001; Ryu and Otsuki,2002; Monteiro and Ryu,2004; Ryou and Otsuki,2005; Jiang et al.,2008; Zhu et al.2014;Yan et al.,2013) have been conducted on the EDM, which can be mainly classified into two categories. The first class is about how to simulate cracks in reinforced concrete (RC) in the laboratory. To simulate the damaged RC, different methods have been adopted to produce the so-called cracks, such as cracks due to a specified load (Ryu,2003a; Monteiro and Ryu,2004; Ryou and Otsuki,2005; Mohankumar,2005), cracks due to drying shrinkage (Otsuki and Ryu ,2001; Ryu,2001) , cracks induced by carbonation (Ryu and Otsuki ,2001), cracks due to chloride attack (Ryu and Otsuki,2002) and artificial cracks (Chang et al.,2009). However, these methods have several shortcomings such as crack randomness, out of controlling of width and orientation of cracks, and difficulty in evaluating healing effectiveness of cracks (Jiang et al., 2008). Therefore, according to the author's previous work (Jiang et al.,2008; Zhu et al.2014;Yan et al.,2013) porous concrete specimen with the steel bar as the negative electrode were utilized to simulate damaged concrete in order to evaluate the healing effectiveness of EDM effectively.

The second category is mainly about the method's assessment and the factors influencing the healing effect. The types and concentrations of the solutions, the density of the current, the properties of the concrete and the temperatures are the main factors that influence the healing effectiveness (Otsuki and Ryu,2001; Ryu,2003b; Chu and Jiang,2009).The most common approaches to assess the effectiveness of the EDM are the specimens' weight gain, the closure of cracks, the cracks' filling depth, and the concrete's water permeability (Otsuki, et al.,1999; Ryu and Otsuki,2002; Ryu,2003b). When the porous specimen repaired by EDM are considered, a series tests are performed to evaluate the healing effectiveness in our present work , which include the ultrasonic pulse wave test, the three-point bending test and the Archimedes test.

ELECTROCHEMICAL DEPOSITION TEST PROGRAM

Preparation of specimens. The cement used is Grade-52.5 ordinary Portland cement produced by Huaxin Cement Group. Aggregates with size of 2.5-10 mm are used to prepare the porous specimen. Finer aggregate with size less than 2.5 mm is not employed. Chinese Industrial Standard deformed steel bars are used, whose diameters are 8 mm. The aggregate-cement ratio is 0.22. Water-cement ratio is 0.3.

The dimensions of the specimen are 100×100×300 mm with a cover depth of 20 mm. The reinforcement cages using steel rebar are fixed in middle of the specimens. The specimens are cured at the standard curing room with temperature: 20 ± 3℃, and relative humidity (RH): >90%, which are taken out of the molders after 48 hours curing and are cured continuously for 28 days. Figure2 shows the steel bar location and sensor Location of Pundit Lab ultrasonic device. Figure3 displays the porous specimen after healing, where the white part is the deposition products.

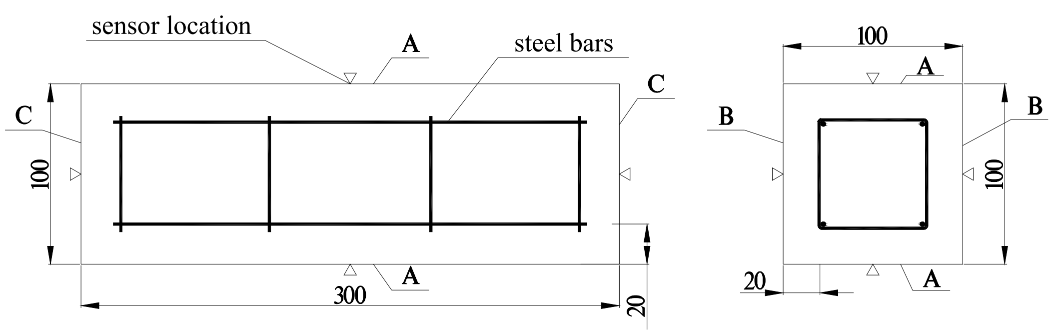

Figure 2. The steel bar location and sensor Location of Pundit Lab ultrasonic device.

Figure 3. The porous specimen after healing (the white part is the deposition products).

Application of electric current and immersion solution. Direct current is fed by a power supply between the embedded reinforcing steel and a titanium mesh immersed in the solution and located at the bottom of the container. The embedded steel is connected to the negative terminal of the power supply and the titanium mesh is connected to the positive terminal. The current density is kept 0.2A during the healing process. The concentration of $Mg(NO_3)_2$ solution is 0.25 mol/L. To keep the concentration constant, the solution is changed every two weeks.

TEST ARRANGEMENT

The three-point bending tests, Archimedes tests and the non-destructive tests using the ultrasonic pulse velocity (UPV) are performed to obtain the evaluation indexes.

Ultrasonic pulse velocity test. The Pundit Lab ultrasonic device is employed to perform our experiment. The wave velocity R is calculated using the time taken by the pulse to travel the measured distance between the transmitter and the receiver as $R = L / t$, where L and t are the distance between the two transducers and the pulse traveling time, respectively. Since the rebar in the specimens may cause error to the results we need, the testing point of ultrasonic probe is carefully selected in the middle of every couple of parallel surfaces. See details in Figure2. The pulse velocities between three couples of parallel specimen surfaces are tested after 0, 14 and 35 days healing.

Three-point bending test. To study the influence of the EDM on the mechanical performance, the three-point bending test of the healed and unhealed specimen are measured in accordance with Chinese test standard GBJ 81-85.

Archimedes test. To monitor the porosity evolution of the porous specimen, we use the porosity obtained by Archimedes method to evaluate the healing effectiveness (Jiang et al., 2008), which can be approximately reached by $P_T = (B - A) / (\rho_w V) \times 100\%$, with A and B representing the oven-dry mass（80℃ for 24 hours） and the mass after immersion i.e. the underwater weight of specimen; ρ_w is the density of water; V is the volume of the specimen. It is noted that absorptions in aggregate and paste are not considered in the calculation.

TESTING RESULTS

Ultrasonic pulse velocity. The average UPVs increase from 3546m/s before healing, to 3617 m/s after 14-day healing, to 3656m/s after 35-day healing. However, when different surfaces or specimens are considered, their UPVs are not exactly the same during the healing process, as shown in Figure2. The increase of UPV indicates that the compactness of porous concrete improves because the pores are filled by the electrochemical deposition products after healing.

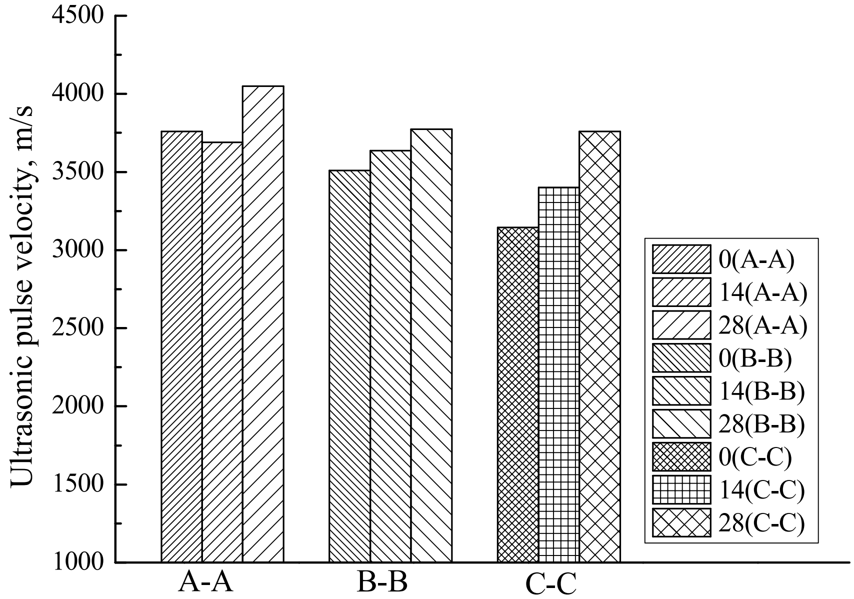

Figure 4. The UPV evolutions of the individual specimen (where A-A, B-B, C-C represent three different parallel surfaces, as shown in Figure2).

The flexural strength. It can be found from Figure5 that the flexural strength of the porous specimens can be improved after the EDM test, with an average increase of 15%. Similar results can be found from the works of Ryou and Otsuki(2005); Chang et al.(2009). Their results both indicate that the EDM can improve the flexural strength by 20%. The main reason is that the (micro-) pores or cracks are repaired by the electrochemical deposition products.

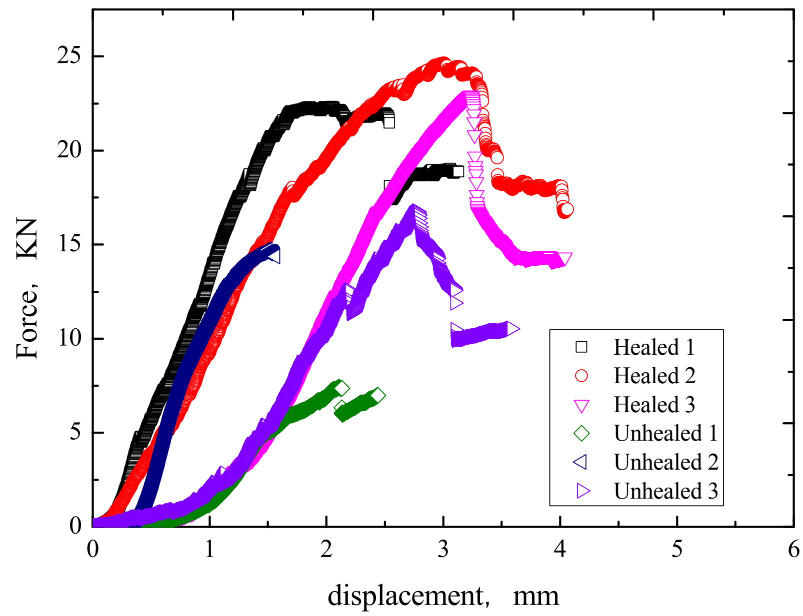

Figure 5. The relation between the displacement and force of three-point bending test.

The porosity evolution. Table 1 presents the results of the Archimedes tests. The specimens' porosities decrease during the process. The reason for the porosity reduction is due to the fact that the pores are filled by the deposition products during the healing process. The ensemble average porosities reduce from 0.2999 before healing, to 0.26245 after 14-day healing, to 0.24092 after 35-day healing.

Table. 1 The evolution of porosities during healing process.

Healing Days \ Samples	1	2	3	4	5	6	7
0	0.271	0.276	0.334	0.311	0.320	0.306	0.280
14	0.233	0.249	0.315	0.282	0.279	0.268	0.230
35	0.214	0.220	0.285	0.252	0.258	0.267	0.213

Healing Days \ Samples	8	9	10	11	12	13
0	0.326	0.296	0.322	0.243	0.313	0.300
14	0.303	0.224	0.297	0.223	0.272	0.237
35	0.272	0.210	0.269	0.240	0.247	0.240

CONCLUSION

In this paper, considering the fact that there are several shortcomings in producing the cracked specimen, such as crack randomness, out of controlling of the crack width and orientation, the porous concrete specimens are employed to perform the EDM experiment. The following conclusions can be obtained from our present work.

(1) The EDM is feasible for the rehabilitation of deteriorated reinforced concrete, which lays the groundwork for research on its practical application on real structures.

(2) Since the deposition products can fill the pores of specimen, the damage of the samples can be healed. The UPVs and the flexural strength of porous specimens increase while their porosities reduce during the healing process.

It is noted that the progression of hydration in the samples with age may contribute to the strength gain, which will be studied in our future work. The factors that influence the healing effectiveness are not studied. The healing effect will be different when changing certain settings like the specimen type, the current density and so on. For example, the deposition velocity may increase but the electrochemical deposition products may become loose when the current density is high, see details for the work of Jiang et al (2008) and Ryu (2003b).

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Field Evaluation of Precut Thermal Cracks in an AC Pavement in Alaska

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Abstract

Thermal cracking is a natural feature of most of Alaska's asphalt concrete (AC) pavements that influences long term maintenance costs and drivers' perception of road performance. Major transverse thermal cracks penetrate through not only the pavement layer itself but usually extend several feet into the aggregate materials below. A significant portion of Department of Transportation (DOT) Maintenance and Operations budget has been allocated to crack sealing and associated work. However, Alaska researchers are beginning to understand that the inevitable thermal cracking can be significantly controlled by making simple changes to new road designs. Past research in Alaska and elsewhere found that thermal cracks can be controlled if properly spaced saw cuts are made in the pavement surfaces of newly built pavement structures. Research presented in this paper looks at the different case of whether precutting can influence thermal cracking when only the top few inches of an old, thermal cracked, pavement structure have been reconditioned and repaved. Two years of monitoring show that precutting exerts significant control on thermal cracking of the new pavement surface even if most of the previously-cracked underlying aggregate pavement structure was left in place. However, precutting is most effective when the saw cuts are made at or very near the locations of the old thermal cracks.

INTRODUCTION

Road-width thermal cracks (major transverse cracks) are perhaps the most noticeable form of crack-related damage on asphalt concrete (AC) pavements throughout colder areas of Alaska. In these colder regions, it has not been possible to prevent this crack type from forming. To date, this appears to remain true regardless of paving material, embankment material, or construction method.

The practice of precutting in pavements is not a new one (Chojnacki 2001). Contraction or control joints are generally established in concrete pavements through precutting (sawing) of the hardened concrete. These sawed joints create a weak plane in the slab, which promotes cracking of the slab at that plane. The joints provide the stress relief needed for the concrete while controlling the location of cracks. Studies have been conducted to improve construction (procedures, and the depth and timing the saw) (Gaedicke et al. 2007), and evaluate long term performance and of concrete pavements with joints cut with early-entry saws (Krstulovich Jr et al. 2011). Another application of precutting is to control the reflection cracking above the joints when asphalt overlay is placed to concrete pavements (Wojakowski and Catron 1995). However, very limited research has been conducted to evaluate its feasibility in AC pavements.

In 1984 an Alaska research project began to study the problem of transverse thermal cracks in a practical way. The basic idea behind this project was that if the thermal cracks could not be prevented, then perhaps it would be possible to create a more acceptable form of transverse thermal cracking. An Experimental Feature research project was started at that time to investigate the possibility that “better” thermal cracks could result if the location and geometry of the crack could be controlled by precutting a thermal crack pattern in a new pavement. The technique, applied on Phillips Field Road (near Fairbanks’ urban center) in October of 1984, consisted of cutting thin slots through the pavement to within about ¼ inch of the bottom of the new asphalt concrete layer. The thin saw cuts were made perpendicular to the road’s centerline and from pavement edge to pavement edge. Spacing between presawn thermal cracks was set at 50 feet. The 50-foot precutting interval was chosen because it was the average of many measurements of natural thermal crack spacings from much research work previously done in interior Alaska. Unbeknown to Alaska researchers at the time the test section was proposed, the Minnesota had experienced success with presawn thermal cracks since first testing the technique in 1969 (Morchinek 1974). Today, after more than 30 years, the Philips Field Road precut section remains in very good condition. In a nutshell, the presawing done on the heavily trafficked Phillips Field Road has been an unqualified success. Some of the presawn cracks became active thermal cracks (as evidenced by significant subsequent movement) and some did not. Of the presawn cracks that did not become active thermal cracks, the presawing did no long term harm to the pavement; the precut lines are visible, but there has been essentially no degradation of the pavement adjacent to the original precut lines.

Regarding some of the practical benefits of precutting, it is obvious that the better appearance of precut transverse cracks, especially in urban areas, provides the impression that the pavement has been more “professionally finished.” Regardless of other benefits, drivers will perceive the precut pavement as having a much less “damaged” appearance, and that it is therefore less in need of maintenance than if naturally cracked. Also, authors’ subjective conclusion based on driving the Phillips Field Road experimental section innumerable times since 1984 is that vehicle ride smoothness has been preserved due to: 1) less spalling of natural cracks and 2) less accumulated maintenance material. Phillips Field Road became the showpiece example of long-term success for the precutting method that finally prompted new precutting experiments in interior Alaska. Past research in Alaska and elsewhere found that thermal cracks can be controlled if properly-spaced saw cuts are made in the pavement surfaces of newly built pavement structures.

Research presented in this paper looks at a different design/construction situation. The research addresses the question of whether precutting can influence thermal cracking when only the top few inches of an old, thermal cracked, pavement structure have been reconditioned and repaved. A construction project located near Fairbanks was selected for this study. During the 2012 construction season, transverse crack precutting was done within a 1-mile section of the Alaska Department of Transportation and Public Facilities (AKDOT&PF) Richardson Highway Mile 340 to 346 resurfacing project. Results from field crack surveys were summarized and analyzed. Conclusions and recommendations for further research were presented as well.

CONSTRUCTION

Experimental saw cutting (precutting) was done as part of the Richardson Highway MP 340 to 346 resurfacing project. A mile-long experimental section was defined on the project as starting about 16 ½ miles highway miles southeast of Fairbanks of the Richardson Highway and extending about 1 mile southeast along the highway in the southbound lane.

Figure 1. Precut work in the field.

The experiment was composed of four subsections: Section 1 which is the control section without saw cuts; Section 2 which has 17 cuts of each of the three depths (0.5", 1.0", and 1.5") with each 25' apart; Section 3 which has 11 cuts of each of the three depths (0.5", 1.0", and 1.5") with each 40' apart; and Section 4 which has total 28 cuts (7 at a depth of 0.5", 10 at 1.0" and 11 at 1.5") and the cuts are located over the cracks in the asphalt that was replaced (the preconstruction natural thermal cracks). Precut work was performed by an employee of the contractor on the southbound lanes (Figure 1). During cutting, traffic control consisted of closing a single lane of the two south bound lanes. The single lane closure allowed cutting of approximately two thirds of the 2-lane width from one side of the road. After all cuts were partially completed, the lane closure was switched to the adjoining lane to allow completion of the saw cuts. The cuts extended from edge of pavement to edge of pavement (full 2-lane width) at each cut location. Saw cutting of the 111 full-width slots required three full workdays. Weather during the sawing operation ranged from partly cloudy to rain, with temperatures between 50 and 80 degrees F.

CRACK INSPECTION AND RESULTS

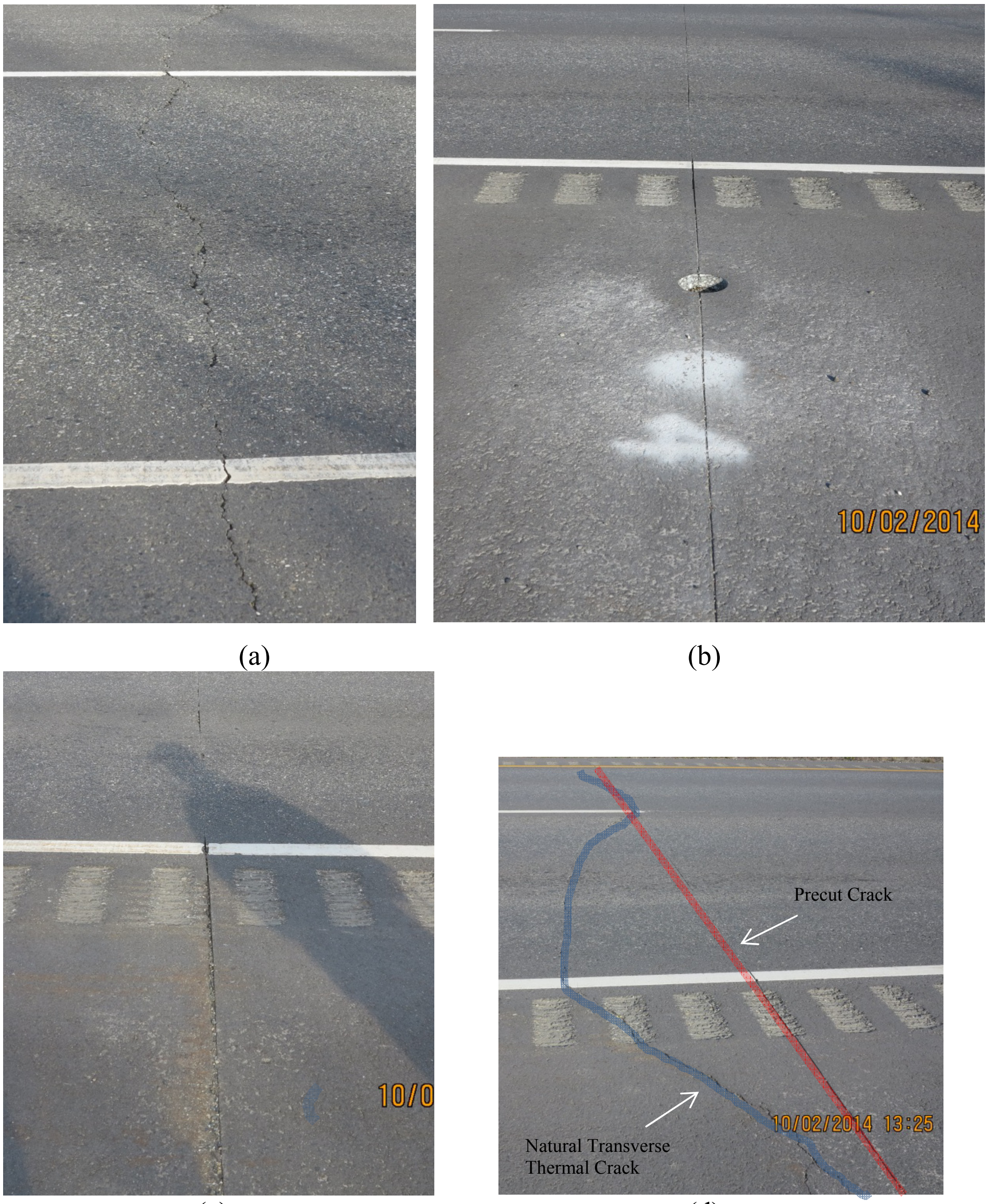
A general visual inspection of the four research subsections was done on October 2, 2014, about two years following resurfacing. Photos were obtained to document the various kinds and condition of cracking that had developed. This inspection revealed the presence of several characteristic crack types described as follows.

Natural transverse thermal cracks (natural cracks) are the natural cracks that developed completely independent of any precutting, i.e., transverse thermal cracks as would be found on any other paved road in the general area (Figure 2a). It may be of interest to learn that natural transverse thermal cracks also commonly form in gravel roads. Evidence of such cracking is fleeting however because movement of loose aggregate surfacing material tends to fill and/or cover obvious signs of thermal cracking at the gravel road surface.

Precut transverse cracks—non-active (precut) are precut transverse cracks that have not been activated by intrusion of the natural thermal cracking process. In other words, a non-active precut crack is simply a slot that has been saw-cut across the pavement surface. In all precutting experiments done to date in Alaska, saw-cuts have not extended completely to the bottom of the pavement layer (Figure 2b).

Precut transverse cracks—active (precut active) are precut transverse cracks that have been activated by the natural thermal cracking process. After activation by the natural cracking process, these cracks become, in effect, simply man-influenced natural thermal cracks (Figure 2c). The experimental section's pavement is only two years old at the time of this reporting, and at this time it is nearly impossible to visually differentiate between non-active and active precut cracks. Differences will likely become more perceptible in later years as active cracks mature and become obviously wider after periods of low temperature. Also, a narrow zone of pavement adjacent to (paralleling) a mature active crack should become slightly depressed with time. In the case of a new pavement, the only sure way of discriminating between active and non-active precut cracks is to make repeated measurements of crack width. The widths of active cracks will cycle with long term temperature variations. While width variations of active cracks in recently constructed pavements may be slight, experience in the Fairbanks area indicated that annual width variations of ½ inch or more may be common for older pavements (McHattie 1980).

Precut transverse cracks with partial capture of natural transverse cracks (precut partial) are by far the most interesting cracking type seen within the experimental subsections. Such precut cracks appear to be partially active and partially non-active—both conditions in the same crack. In instances where natural cracking occurs fairly close to a precut crack (apparently within about 4 to 6 feet), one or more portions of the natural crack may intersect with the precut crack and become integrated with it for some portion of the precut crack's length. Figure 2d shows the location where the precut crack has partially "captured" the natural crack. Some of the partial capture occurrences appeared to be interestingly complex, where the natural crack entered and exited the precut crack sometimes two or, in one observed case, three times.



(a) (b)

(c) (d)

Figure 2. Types of cracks inspected: a) natural transverse crack, (b) precut non-active crack, (c) precut active crack, and (d) precut cracks with partial capture of natural cracks.

Resurfacing construction on the Richardson Highway MP 340–346 project required only surficial processing (reclaiming) of the top few inches of the pavement structure. The previously existing pattern of major transverse thermal cracking was allowed to remain in place, eventually covered only by two inches of new asphalt concrete and a few inches of reprocessed old pavement—perhaps 6 inches total new material placed atop the old crack pattern. The crack inspection suggests that—given this type of construction—the desired precutting effect, i.e., complete capture of subsequent natural cracking, requires that the precuts be placed as closely over the previously existing transverse cracking pattern as possible. This was strongly demonstrated in Subsection 4. In Subsection 4 precuts were placed nominally at the locations of the preexisting transverse cracks. Nominally in this case means that each precut in Subsection 4 would be centered at the mid-length point of the old crack, although actually precut perpendicular to the centerline. Thus the new precut did not exactly follow the existing thermal crack if the existing crack had a complicated shape and/or was strongly skewed to the centerline. In Subsection 4 most of the precut cracks seem to have either partially or fully captured the subsequent natural cracks. The assumption at this point is that, had the Subsection 4 precut cracks more exactly traced the existing thermal cracks, the success rate of total captures for the precuts would have been much higher.

CRACK SURVEYS AND ANALYSIS

Crack surveys were performed on October 22nd, 2013 and April 24th, 2014. The crack surveys required measuring the distance of every visible major transverse thermal crack from the reference location at Station 989+95. These measurements were done using a surveyor's "walking wheel," with a precision of about ± 2 foot over the mile-long experimental section. These crack location determinations were made while walking in the right shoulder of the southbound lanes. Most of the transverse cracks were found to be skewed to the roadway centerline. Therefore, the location of each transverse crack was noted on the field data sheet as the location of the right end of the crack. Also noted was whether the right or left end of the crack was skewed forward ("right forward" or "left forward" skew) or not skewed. Thus individual natural cracks were classified as either a right forward skew type, a left forward skew type, or a no-skew type. The locations of all precut cracks were also determined as part of the 2013 and 2014 surveys. This was done, in part, for the purpose of verifying that the walking wheel was giving accurate locations over the entire survey mile. All precut cracks were found to be at the locations listed by AKDOT&PF engineers after construction.

Table 1 summarizes natural crack spacing and counts from field surveys. The term "natural crack" refers to those transverse cracks that extend across the full width of the paved surface and are not precut. Precut cracks that have already (or will) become active are certainly involved in the natural cracking process, but they are not considered as natural cracks per se. In this study, precut cracks—whether active or not— are considered to be a pavement design feature and not a form of damage. The purpose of this analysis is to determine the extent to which various precut designs are able to limit, i.e., control development of natural thermal cracking not associated with the precut cracks.

Table 1. Natural Crack Spacing and Counts from Field Surveys.

	Preconstruction Natural Cracking	Post-Construction Natural Cracking	
		2013	2014
Average Natural Crack Spacing (All subsections)	72.9	67.1	55.4
Standard Deviation of Spacing (All subsections)	32.9	46.4	41.8
Total No. of Natural Transverse Cracks (All subsections)	74	81	98
Average Natural Crack Spacing (Subsection 1)	60.8	42.1	34.4
Standard Deviation of Spacing (Subsection 1)	21.4	9.6	10.6
Total No. of Natural Transverse Cracks (Subsection 1)	22	32	39
Average Natural Crack Spacing (Subsection 2)	80.8	77.4	58.1
Standard Deviation of Spacing (Subsection 2)	30.3	42.9	39.0
Total No. of Natural Transverse Cracks (Subsection 2)	17	17	22
Average Natural Crack Spacing (Subsection 3)	74.5	71.7	60.8
Standard Deviation of Spacing (Subsection 3)	33.2	36.6	29.7
Total No. of Natural Transverse Cracks (Subsection 3)	17	18	22
Average Natural Crack Spacing (Subsection 4)	75.6	101.2	96.5
Standard Deviation of Spacing (Subsection 4)	39.0	78.4	70.8
Total No. of Natural Transverse Cracks (Subsection 4)	18	14	15

It can be seen from Table 1 that before 2012 construction, the number of natural transverse cracks in all four subsections was similar — 22, 17, 17, and 18 in subsections 1 through 4, respectively. In just the two years since construction, the crack counts in subsections 1, 2, and 3 had actually increased by 77%, 23%, and 17% over the preconstruction number, respectively. Only the natural transverse crack count in subsection 4 remained lower than the preconstruction number—17% lower than the count prior to the repaving job. The subsection 4 precut design approach appears quite superior to those used in subsections 2 and 3.

Table 2 shows how precut depth influences precut effectiveness (for a 2 inch total pavement thickness). Although Table 2 provides no definitive degree of evidence, it appears that the areas precut to depths of 1 inch or 1 ½ inch produced fewer natural transverse thermal cracks. By 2014, the 0.5 inch precut depth produced the highest number of natural transverse cracks in all precut subsections. The table suggests that there is some degree advantage to deeper precuts although there is no evidence that a 1 ½ inch cut depth is better than a 1 inch cut.

Table 2. Influence of precut depth on observed natural cracking (total pavement thickness = 2”).

	Natural Crack Count during Indicated Survey Years	
	2013	2014
Subsection 2		
0.5” Cut Depth	7	9
1.0” Cut Depth	3	5
1.5” Cut Depth	6	7
Subsection 3		
0.5” Cut Depth	5	9
1.0” Cut Depth	5	6
1.5” Cut Depth	4	5
Subsection 4		
0.5” Cut Depth	6	6
1.0” Cut Depth	4	4
1.5” Cut Depth	4	4

In brief summary, the preconstruction thermal cracking condition of all of the four subsections was similar, i.e., before resurfacing construction and precutting. The control section has performed very significantly worse than the three precut subsections in terms of the appearance of new natural transverse thermal cracks. Except for subsection 4, the count of natural transverse thermal cracks now is higher than it was before the 2012 construction project. Subsection 4 obviously exhibited the best thermal cracking performance to date. All precuts in this subsection were placed at the approximate locations of preconstruction natural cracks. There is a tenuous indication that the precut crack depths of 1 inch and 1 ½ inch have worked better than those of ½ inch depth (pavement total thickness = 2”).

CONCLUSIONS AND RECOMMENDATIONS

Precutting technology has been shown to work well in cases where roadway construction has included placement of at least several feet of new material. This has been demonstrated in Minnesota (Morchinek 1974, Janisch 1996) as well as by the 30-year-old test section at Fairbanks, Alaska. With the caveat that the Richardson Highway experimental section reported herein has been monitored for only two years, this research tentatively indicates that precutting can significantly benefit the thermal crack performance of a pavement resurfacing project.

The best performing experimental precut subsection was where each precut was placed at the location of a transverse thermal crack that existed prior to reconditioning and repaving. This makes sense according to the literature review and in view of long term observations at many locations in Alaska. Many years of Alaska experience has confirmed that full-width "major" transverse thermal cracks extend into the aggregate materials as much as several feet below the bottom of the asphalt concrete pavement. In Alaska it is known that the pattern of transverse thermal cracking continues to exist within underlying materials whenever construction involves only the upper few inches of an existing, thermal cracked pavement structure.

Findings presented in this paper were based on preliminary results from a relatively short time period. Continuing to survey and monitor these four subsections is recommended. Careful measurements of width variations of the precut slots throughout at least one annual temperature cycling would be required to define which precuts have become active. It would be helpful to compare/correlate future findings from precutting subsections with field practice of crack sealing and recommend an effective design methodology and construction practice to control thermal cracking in AC pavements for Alaska and cold areas of other northern states.

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Improved User Experience and Scientific Understanding of Anti-Icing and Pre-Wetting for Winter Roadway Maintenance in North America

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Abstract

In recent years, North American highway agencies have increasingly adopted innovative snow and ice control strategies, including anti-icing and pre-wetting practices. In 2005, to synthesize the benefits and challenges of implementing such strategies, a national survey was conducted. However, insufficient information and limited understanding were found to account for their low usage at that time. A decade later, research is needed to document improved user experience and scientific understanding on these strategies, which can be used in a comparative manner. Through a follow-up survey launched in April 2014 with the same focus as the 2005 survey, a contrastive synthesis is presented which summarizes changes in the evolution of anti-icing and pre-wetting over the past decade, including the implementation of these snow and ice control strategies, related costs, environmental concerns, corrosion, and community response. The research demonstrates the elevated and indispensable status of anti-icing and pre-wetting strategies in current winter roadway maintenance activities, compared with their limited use ten years ago. The research further supports their positive effects on economic, environmental and social goals, which is attributable to the proactive nature and environmentally responsible performance of anti-icing and pre-wetting. Finally, gaps in current operations and the need for further research are discussed.

INTRODUCTION

Winter roadway maintenance agencies in North America have multiple objectives in planning and undertaking snow and ice control activities during winter seasons, including traveler safety, environmental stewardship, infrastructure preservation and economics (O'Keefe and Shi, 2006). The tradeoff in balancing these concerns calls for the development and adoption of advanced materials, techniques and treatment strategies in winter maintenance operations. As promising and efficient snow and ice control strategies, anti-icing and pre-wetting are considered as a positive development in winter maintenance.

The term *anti-icing* refers to early application of chemicals before inclement snowfall to prevent black ice and weaken the bond between roadway surface and icy snow, while *pre-wetting* adds liquid (e.g. hot water or chemicals) to solid chemicals or abrasives before or during application to increase the effectiveness of solid materials (Sooklall, Fu and Perchanok, 2006; Shi et al., 2013a). Typically pre-wet solid salt or abrasives are used for deicing and sanding operations, respectively. The application of pre-wet solid salt for anti-icing, while feasible, has not been as popular as the application of liquid salt brine for anti-icing.

Compared with other conventional reactive snow and ice control strategies (e.g. snow-plowing, sanding, and de-icing), anti-icing and pre-wetting treatments have much in common: (1) they both work proactively for maximizing the consequences with less consumption in a fiscally and environmentally responsible manner; (2) they both typically use liquid treatments due to the high efficiency of pre-treated materials; (3) they both have some constraints for implementation, e.g., under the conditions of inappropriate climatic conditions, improper material handling, and unskilled operator, anti-icing and pre-wetting could make the roadway situation less safe for public travelling than the traditional strategies. Based on these commonalities, this work will examine anti-icing and pre-wetting together.

In view of the merits and drawbacks of anti-icing and pre-wetting for winter maintenance, a survey was conducted in 2005 to identify and synthesize information on using these improved methods. The survey focused predominately on the states and provinces that participate in the Pacific Northwest Snow fighters Association (PNSA). Possibly due to insufficient information and limited understanding, the implementation of anti-icing and pre-wetting back in 2005 was not as frequent and widespread as snow-plowing, sanding and de-icing. After a decade's development in the field of winter maintenance from 2005 to 2014, there is a need to document the changes that have occurred in current winter maintenance practices in North America. There are many factors that played a significant role over this period in the development of anti-icing and pre-wetting practices, such as advances in Road Weather Information Systems (RWIS) and customized weather services (Abdi et al., 2012; Ye et al., 2009a), in-vehicle technologies and enhanced snowplows (Ye et al., 2012; Fay et al., 2010), and Maintenance Decision Support System (Ye et al., 2009b), the AASHTO anti-icing/RWIS computer-based training, and enhanced understanding of the best practices for environmental sustainability (Fay and Shi, 2012; Van Meter et al., 2011), etc. This work will put a focus on the implementation of anti-icing and pre-wetting strategies and summarize the state of knowledge and best practices in anti-icing and pre-wetting strategies for winter roadway operations. A practitioner survey in 2014 on anti-icing and pre-wetting practices will be presented, including a comparative data analysis with the 2005 survey results. Changes in the current practice of winter maintenance, pre-wetting and anti-icing strategies, and related issues will be synthesized. This work will conclude with a brief discussion of future research directions and conclusions.

LITERATURE REVIEW

There is growing research interest in how to maximize the advantages of anti-icing and pre-wetting strategies and reduce resulting negative impacts to the surrounding environment, due to their widespread application for winter maintenance. Every year tons and gallons of anti-icing chemicals are applied on the road surface to depress freezing point of snow-salt mixture according to the salt requirement of the adopted chemical's freezing temperature. However, Klein-Paste and Wåhlin asserted that the required amount of salt in practice is less than the amount predicted by the theoretical freezing curve as anti-icing chemicals could weaken the formed ice, and the traffic could help to destroy the weakened ice. In light of this viewpoint, an alternative physical mechanism was provided in Klein-Paste and Wåhlin's study to examine the ice weakening process under the presence of anti-icing chemicals. A minimum brine fraction of 0.4 was also proposed to show the minimum amount of salt needed in practice (Klein-Paste and Wåhlin, 2013). Different from many laboratory and field testing experimental studies, this research focused on investigating the inherent working mechanism of anti-icing chemicals. It represents one of the new types of anti-icing studies that have extended scientific understanding in the last few years.

For anti-icing and pre-wetting operations, material selection and evaluation is always one of the primary considerations. The selection and use of anti-icing and pre-wetting materials need to take into account many factors, including lowest melting temperature, cost, availability, and environmental impacts (MPCA, 2008). O'Keefe and Shi (2006) listed some detailed specifications developed by PNSA to provide guidance for maintenance agencies in the selection of snow and ice control chemicals. For example, constituent limits in parts per million (ppm) of chemical products, the required analyses for liquid products, and additional analyses for new chemical products were summarized in this study. Fay and Shi (2012) provided a comparison table to identify the defined heavy metals of interest and their total allowable limits in snow and ice control products specified by the Colorado Department of Transportation (DOT) and PNSA. A series of performance evaluation methods were also developed to assist in the selection of anti-icing and pre-wetting materials from environmentally sustainable or anti-corrosion perspectives (Shi et al., 2012, 2013b). For example, Shi et al. (2014) presented a comprehensive and quantitative evaluation method for the chemicals used by Idaho DOT to identify the most sustainable materials by using the laboratory and field test data and reasonable assumptions. Muthumani et al. (2014) developed a laboratory oratory test that could correlate the field test results with information from practitioner interviews to better simulate anti-icing chemical performance. Based on these study results and practical evaluation, researchers have established that numerous products have been adopted for anti-icing and pre-wetting, in addition to traditional chloride-based salts. These products include a number of organic-based alternatives and agricultural byproducts derived from corn, beets and grains, such as acetates (e.g., calcium magnesium acetate and potassium acetate), glycols, formats (e.g., sodium formate and potassium formate), particularly for some critical and environmentally sensitive areas (e.g. airline industry, bridges and other structures sensitive to chloride corrosion) profiting from their non-corrosive and biodegradable properties (Fay and Shi, 2012; Fu et al., 2012).

The effectiveness of pre-wetting not only depends on the selected materials, but also has much to do with the treatment techniques. Michigan DOT examined the differences of various pre-wetting treatment influencing factors on the amount of salt bounce and scatter, such as effective truck speed and delivery system (MDOT Operations Field Services Division, 2012). It was concluded that the most effective way to deliver a pre-treated salt product is through the use of a vehicle traveling at 25 mph with a truck mounted cross conveyor.

Using anti-icing and pre-wetting chemicals induces environmental impacts similar to de-icing and sanding treatments, since they all have negative impacts on the receiving roadside soil, water bodies, aquatic biota, and vegetation through snowmelt runoff, infiltration and wind blow (Todd and Kaltenecker, 2012; Perera et al., 2013), as well as motor vehicles (Shi et al., 2009a), and transportation infrastructure (Koch et al., 2002; Shi et al., 2009b, 2010a, 2010b, 2011). However, by using liquid materials, anti-icing and pre-wetting can help to reduce application rates and material usage, and thus reduce detrimental impact to the environment. So far, there are few studies that have tried to directly compare the environmental impact of liquid and solid snow and ice control products due to the numerous unquantifiable parameters in the receiving environment. But it is recognized that the liquid chemicals are more concentrated at the beginning of application, and as time goes on, their influence weakens quickly through dilution and runoff; while solid chemicals can maintain a high level of concentration even after a certain period of application (e.g. 60 min) due to the slow release process, and the retention of solid materials would continue to affect the surroundings for a longer time.

This work will focus on anti-icing and pre-wetting strategies in current winter roadway maintenance operations, to document improved understanding that has developed over the past decade, to identify their benefits and existing limitations in application, and to provide information and guidance for maintenance agencies and transportation officials.

METHODOLOGY FOR ONLINE SURVEY

The online survey on the practice of anti-icing and pre-wetting for winter maintenance in North America was conducted using the Survey Monkey web site. The survey occurred in March 2014. An extensive number of state departments of transportation, county and city divisions of transportation, township road department, Canadian provincial ministries of transportation, highway administration and maintenance divisions, and companies from consulting, highway infrastructure, highway operation and sustainable salting solutions participated in this survey. The 2005 survey only had fifteen states participating, i.e., Alaska, Alberta, British Columbia, Colorado, Idaho, Minnesota, Missouri, Montana, Nevada, New York, Oregon, Vermont, Washington, Wisconsin, and Wyoming. In contrast, the respondents of this 2014 survey cover a much wider scope, including Alaska, Alberta, Colorado, Connecticut, Idaho, Illinois, Iowa, Kansas, Kentucky, Maryland, Maine, Massachusetts, Michigan, Minnesota, Nebraska, New Jersey, New

Hampshire, New York, North Carolina, North Dakota, Ohio, Pennsylvania, Tennessee, Utah, Washington, and Wisconsin. The respondents include maintenance operation managers, supervisors, superintendents, directors, coordinators, specialists, engineers, administrative officers, program chiefs, and company owners. Thirty-three of the fifty-seven respondents completed the online questionnaire, and the responses were collected for further analysis and comparison mainly.

DISCUSSION

Current practice of winter roadway maintenance. Every method of snow and ice control has merits for different conditions: anti-icing could be most useful to prevent bonding; de-icing could be most useful when a forecast is wrong and bonding occurs; plowing clears the roadway for the public; and sanding provides traction when ice has formed and temperatures are too low to de-ice. However, each storm event is like a fingerprint, there is no single and ready-made “best” method for all circumstances, so when asking “What do you see as the best method for maintaining safe winter driving conditions”, an uniform answer was given in both 2005 and 2014 surveys that a combination of all snow and ice control tools is the best. Respondent states in the 2014 survey further presented their opinions in this combined “best” method as a function of their geological region. Most northern states (e.g., Maine, Michigan, Minnesota, Ohio, and Wisconsin) choose to follow the mode of anti-icing, snowplowing, deicing and in extreme situations sanding. There is also some states (e.g., Illinois, New Hampshire, and Tennessee) adopting a simpler program which incorporates anti-icing, snowplowing and de-icing, or only anti-icing and snowplowing to reap the benefits from pre-storm anti-icing activities. In contrast, in the regions that have long periods of extreme low temperatures (e.g., Alberta and Alaska), anti-icing has limited usefulness during much of the winter season, and snowplowing and sanding are the “best” activities. Finally, different from the practices back in 2005, pre-wetting salt and sand has become a more common practice.

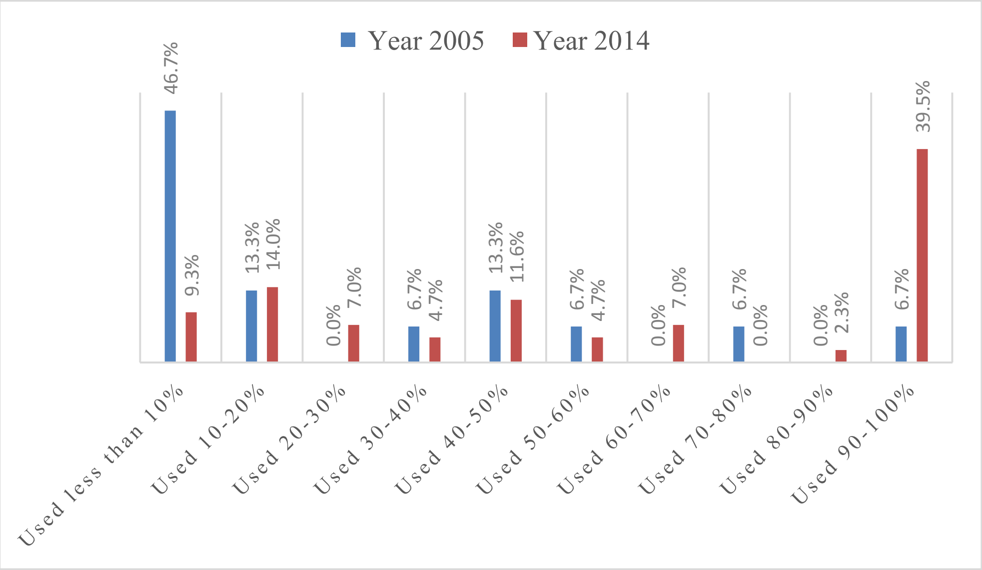

Figure 1(a). Percentage of roadways using anti-icing.

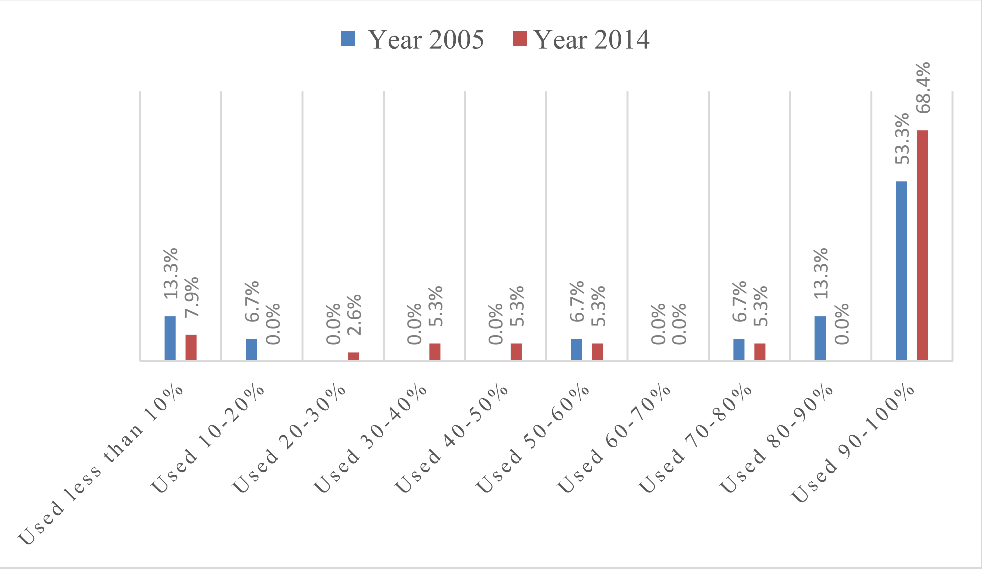

Figure 1(b). Percentage of roadways using de-icing.

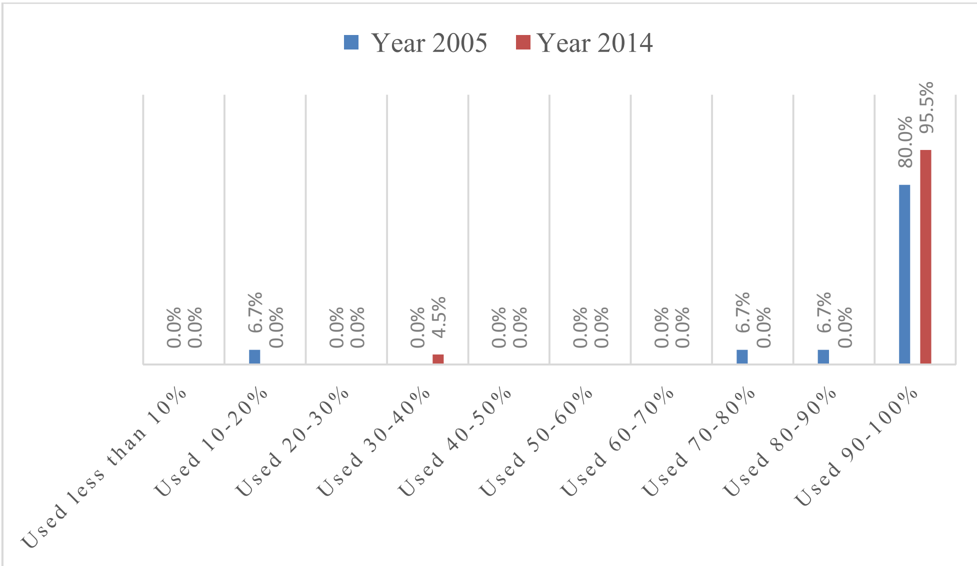

Figure 1(c). Percentage of roadways using snowplowing.

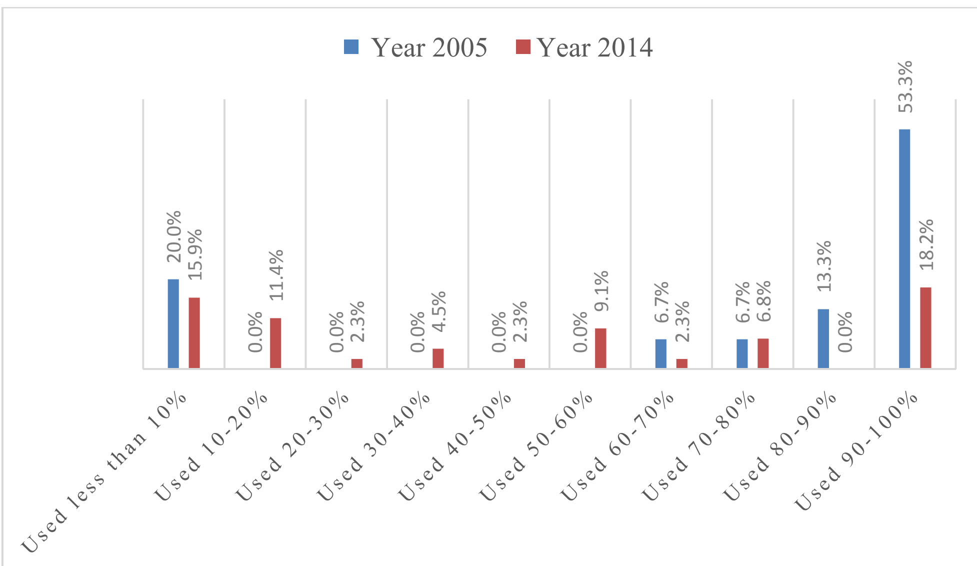

Figure 1(d). Percentage of roadways using sanding.

According to 2005 survey results, snowplowing, de-icing and sanding were traditionally the major winter maintenance strategies across the fifteen respondent states, which was evidenced by a larger percentage of applications showing in blue bars in Figure 1(b), 1(c) and 1(d). In 2005, snowplowing was used 92% of the time, de-icing 76% of the time, and sanding 75%. The same trends are also observed in the 2014 survey, especially snowplowing and de-icing strategies, which on average show that snowplowing is used almost every time (97%) and de-icing maintains a usage around 80%. Comparatively, the use of sanding has decreased by 30%, which is not surprising owing to its negative impacts on the environment. Different from the trends of snowplowing, de-icing and sanding, the results in Figure 1(a) indicate a dramatic change in the use of anti-icing in practices, from less than 10% to the more than 90%, which implies the advantages of anti-icing have attracted great attention from winter maintenance agencies, and anti-icing strategies have gradually become a main component of winter maintenance operations. When calculating the average anti-icing usage across all respondent states, the change from 29% in 2005 to 57% in 2014 further illustrates this point.

It was reported in 2005 that the US spent more than $2.3 billion annually on winter snow and ice control operations. The twelve US states participating in the 2005 survey (excluding Alberta and British Columbia from Canada and Alaska with insufficient information) accounted for approximately 12% of this expense at $24.75 million. After a decade, a sharp increase in snow and ice control expenses at the state level is observed based on the 2014 survey results. The 18 states that provided answers to the cost-related question reported about $60.5 million in average annual costs. These states included Alaska, Connecticut, Idaho, Illinois, Iowa, Kansas, Maine, Maryland, Massachusetts, Michigan, Nebraska, New Jersey, New Hampshire, North Dakota, Ohio, Pennsylvania, Tennessee, and Washington. This large increase in average costs at the state-level also suggests that the total annual cost of US winter maintenance operations has risen greatly. In addition, the results from city, county and township road departments and divisions provide further insights. For example, the responses from cities of Dubuque in Iowa, Columbus in Montana, Fargo in North Dakota, and Columbus in Ohio show that the total snow and ice control cost of big cities in Snowbelt region is around $2 million. While the responses from Mchenry County in Illinois and Otter Tail County in Minnesota indicate that the total cost at these counties is estimated to go averages about $1.5 million. It is thought that approximately $0.2 million is needed at some town levels for snow and ice control operations, when reviewing the responses from Chardon Township in Ohio and Springfield Township in Pennsylvania.

Anti-icing and pre-wetting practice. Both the 2005 and 2014 surveys revealed that most states had a history of using pre-wetting treatments for more than 20 years, and some states (such as Idaho, Minnesota, and Washington) could even track back to the 1980's. However, pre-wetting was not widely used in 2005, as evidenced by less than 30% of respondents used pre-wet materials on average. Based on the responses in the 2005 survey, low usage may have been attributable to the lack of a comprehensive training program for maintenance technicians and equipment constraints. However, the 2014 survey reflects that pre-wetting has become an essential part of winter maintenance. Half of the participant states report 100% usage, and in total with over 80% of respondents report usage, no matter in state, county, city, or township levels. The difference in pre-wetting percentages between 2005 and 2014 is shown in Figure 2.

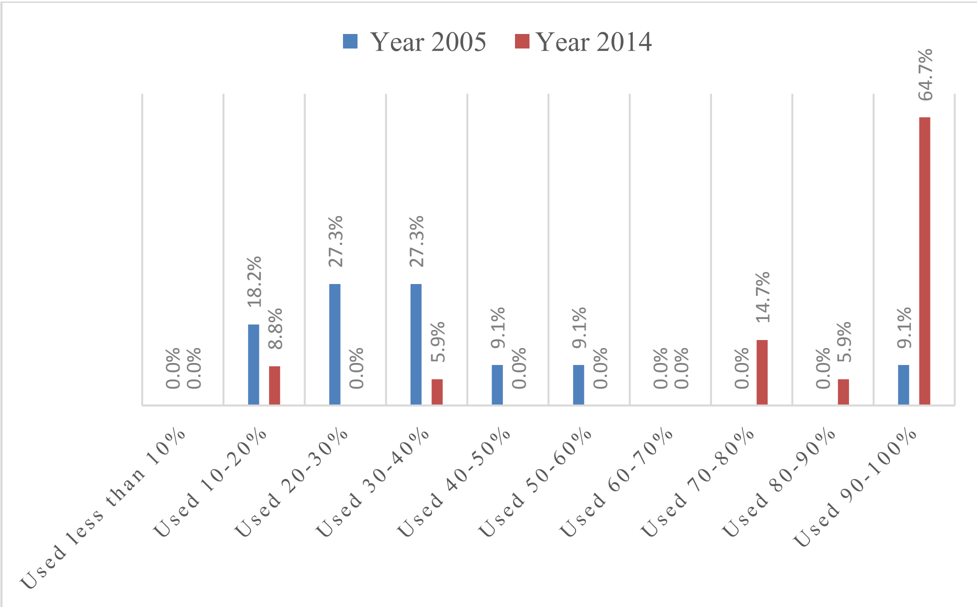

Figure 2. Percentage of sand or solid salts that are pre-wet.

The 2005 survey also stated that about one third of respondent states (including Alberta and British Columbia) had around 20 years of experience with anti-icing, (in other words, they began using anti-icing in the middle of 1990's). These states include Alaska, Colorado, Idaho, Montana, and Washington. The data from the 2014 responses show a similar percentage, 39.1%, and participants reporting long anti-icing experience include Iowa, New Jersey, New Hampshire, Kansas, and Massachusetts. Even so, the responses showed that about 13% of participant states did not start to test and implement anti-icing until after 2004, e.g. Alberta, Michigan and Wisconsin. It was suggested that the widespread implementation of anti-icing has been obstructed by the strong dependency of anti-icing on accurate forecasting and the difficulties in applying the right materials in the right conditions and at the right time. In spite of these limitations, a growing frequency of anti-icing usage is shown in 2014 whenever the weather pattern warrants and adequate information is available from forecasters.

Overview of anti-icing and pre-wetting. Anti-icing and pre-wetting have strong performance on managing roadway safety, mobility and productivity, but also have some detrimental issues, similar to de-icing and sanding treatments. The responses from all participating states in 2014 about the advantages and disadvantages of anti-icing and pre-wetting are displayed in Table 1, and described in more detail below.

When asked "Have the practices of anti-icing and pre-wetting improved roadway safety for your jurisdiction", most agencies claimed that the superiority of anti-icing and pre-wetting in improving roadway safety was witnessed by their high efficiency in achieving bare pavement quickly and by making snowplowing much more easier. As a result, these practices may lead to an increase in service levels, as well as reduced accidents and improved mobility. For instance, in Utah, after anti-icing and pre-wetting applications, the quantity of hard ice and snow pack during storms decreased, and bare pavements post storm were achieved almost three times faster. McHenry County in Illinois found their roadways were much safer compared with previous practices prior to 1995. Alberta has seen network traffic increases of 5% annually for the last decade after they introduced new winter maintenance strategies.

Table 1. Responses about the advantages and disadvantages of anti-icing and pre-wetting treatments in 2014 survey.

	Advantage				Disadvantage	
	Improve roadway safety	Reduce application rate	Economical	Environmental beneficial to Human health	Environmental impact	Vehicle and infrastructure corrosion
AB	Y	N	N	N	Y	N
AL	Y	-	Y	N	N	Y
CO	Y	Y	Y	Y	N	Y
IA	Y	N	Y	Y	Y	Y
ID	Y	Y	Y	Y	N	Y
IL	Y	Y	Y	Y	N	Y
KS	Y	N	N	Y	N	N
MA	Y	Y	Y	Y	Y	Y
MD	Y	N	Y	Y	Y	N
ME	Y	Y	Y	Y	Y	Y
MI	Y	-	Y	Y	N	Y
MN	Y	Y	Y	N	Y	-
ND	Y	Y	Y	Y	N	Y
NE	Y	N	N	Y	Y	Y
NH	Y	N	Y	Y	Y	Y
NJ	Y	Y	Y	Y	N	Y
OH	Y	N	Y	Y	N	N
TN	Y	Y	Y	Y	N	Y
UT	Y	Y	Y	Y	N	Y
WA	Y	Y	N	Y	N	N
WI	Y	Y	Y	Y	Y	Y
Percent (Y)	100%	57.14%	80.95%	85.71%	42.86%	71.43%

The contribution of anti-icing and pre-wetting treatments to reduce the usage of snow and ice control materials is notable. Maine reported their sand usage dropped from around 500,000 CY annually to about 15,000 CY, and they were also able to eliminate over 10% of their working fleet with the usage of anti-icing and pre-wetting. However, relating anti-icing or pre-wetting to reduced application rate is still very challenging, because the data is not always conclusive. In 2005, respondents reported average reductions of 20-30% for sanding applications and about 10% for chemical applications. By contrast, the responses from the 2014 survey were less conclusive, with reductions ranging from as low as 10% to as high as 50%. For instance, Division of Transportation of McHenry County in Illinois reported their reduced application rate yearly by 40% on average. City of West Des Moines in Iowa had a reduction on application rate by almost 30%. Massachusetts DOT has taken out almost 250,000 tons out of the usual applications when compared with similar winter severity indices; and Idaho DOT has reduced application rates down to 150 to 300 pounds per lane mile.

The economic performance of anti-icing and pre-wetting strategies is reflected indirectly by the principle of maximizing results for less consumption, i.e., lowering material usage and saving manpower and equipment hours. Such benefits are usually hard to estimate in a numerical way, but when reviewing the material usage savings, it was estimated that Illinois had a 25% to 30% reduction in materials and Utah had at least a 50% material savings after anti-icing and pre-wetting were implemented. Chardon Township Road Department provided an example on the manpower and equipment savings of anti-icing. Specifically, "when anti-icing we only have one truck on the road, which prevents hard pack and icing, compared to 4 trucks trying to stop hard pack at the time of the event."

There are short-term and long-term risks associated with the detrimental effects of winter maintenance materials. Most participant agencies perceive the negative impacts of pre-wetting and anti-icing on the roadside vegetation, soil, water, and aquatic biota as mild, even though some respondents did report that cases of contaminations had occurred. Corrosion of winter maintenance equipment, public vehicles and concrete transportation infrastructures from the application of dry or wet chloride salts produces effects that are more visible, e.g. $CaCl_2$ and $MgCl_2$. Several participant agencies noted the balance between the premium property of $CaCl_2$ and $MgCl_2$ to lower the working temperature of salt and their negative effects in accelerating corrosion of field assets. Nebraska reported some anecdotal evidence that $MgCl_2$ may accelerate alkali silica reactivity (ASR) in concrete pavements. Massachusetts and Utah noticed the corrosive stress with $MgCl_2$ and $CaCl_2$ in both single and mixed usages. Effective ways to reduce corrosion may include adding corrosion inhibitors in liquids or adopting organic-based additives, and providing good training for operators on proper usage of materials.

Besides the perception of the impacts of direct application of chemicals on the road surface, the leakage in the storage units is another contamination concern. Survey results indicate that solid materials are usually stored in wooden or fabric covered structures (e.g. domes and barns, or tarps), while liquids are stored in bulk containers, of which 5,000 to 10,000 gallon storage tanks are in common use, either with single or double walled, poly or fiberglass materials. However, occasionally salt sheds are damaged in windstorms, UV light degrades tanks as years go by, and liquid recirculation systems and fittings are prone to cause drips and spills, all of which raise the risk of contamination to the surrounding ecosystem, e.g., creeks and wells. In addition, quality control to maintain expected concentration and protect against sludge and sediment at the bottom of tanks is another important storage related concern.

When promoting environmentally friendly winter road service, it is important to consider public perception and public feedback as a mirror to reflect the consequences of maintenance activities. In contrast to the mixed public attitudes towards anti-icing and pre-wetting in the 2005 survey, public perceptions from the

2014 survey are mostly favorable, especially with anti-icing measures, as it is visual evidence that winter maintenance operators were on the roadway and performed preventive snow and ice work. There are occasional complaints that liquid chemicals have caused greater corrosion to vehicles, but the desire for better winter mobility has far outweighed concerns about corrosion, and improved level of service obviously wins more support from the public. Another contributor to the positive feedback is that the use of public education and positive newspaper articles about anti-icing over the years has helped the public understand the importance of anti-icing and pre-wetting. In Tennessee, salt brine/anti-icing has become a warning of an inclement storm. When the public sees TDOT apply salt brine they take action to prepare as individuals, such as fill cars with gas, visit grocery stores, and make arrangements for travel the next day. Schools will also be on notice and close if necessary. In this, it allows TDOT to take care of the roadway and reduce any unnecessary travel when the storm arrives.

Future Research and Practical Innovations. Drawing upon relevant literature, various dimensions of applying anti-icing and pre-wetting strategies have been explored to enhance their treatment performance, especially in the last three decades. Future research may be conducted on the following topics.

1) Physical mechanisms of anti-icing and pre-wetting practices. The existing studies on anti-icing and pre-wetting have put more emphasis on the laboratory and field testing, which can provide findings on the actual performance of treatments to guide practitioners. Nonetheless, the results have a strong reliance on the experimental environment and settings and may not be general. Therefore, more work needs to be done to examine the essential physical working mechanisms. Particularly, the combination of testing and mechanism research will help to promote performance enhancement related improvements in the future.

2) Development of systematic highway winter maintenance mechanisms for anti-icing and pre-wetting strategies. The systematic approach may encompass the following steps: strategic and integrated planning, a life-cycle based sustainable performance evaluation, a comprehensive cost-benefit assessment from purchase, storage, and transportation to final application, a rounded data collection and analysis platform for convenient recording and effective feedback. Systematic approaches may help practitioners to make decisions for anti-icing and pre-wetting strategies from a holistic perspective and reduce potential risks in a proactive way.

3) Exploring the factors that may exert an influence on the development of anti-icing and pre-wetting practices. There are many factors that were responsible for the changes in the evolvement of anti-icing and pre-wetting strategies, e.g., advanced technologies and products, improved operational strategies, and the changing nature of winter storm events. Further exploration into the role of these factors may lead to improved understanding of these practices and facilitate implementation of best practices.

Based on the user experience of survey feedback, many innovations and improvements in anti-icing and pre-wetting practices have been identified as follows.

1) Better and cheaper snow and ice control products. Enhancing the performance of snow and ice control materials, but lowering corresponding cost is

likely to be a continuous requirement for winter maintenance operations. It will be important to find alternatives to chlorides, additives that reduce corrosion, products that lower the effective temperature and provide better staying power on the road surface and less toxicity for the surroundings.

2) Adoption of emerging and improved technologies in winter maintenance equipment. Automated spreaders, improved slurry generators, and expanded usage of automated vehicle location (AVL) will improve efficiency of practice through sensible salting strategies. In particular, slurry technology is attracting more attention as a means of treating multiple types of winter roadway conditions.

3) Better staff training. Usually winter maintenance agencies make drivers of the application trucks responsible for the amount of de-icing chemicals, and conducting correct application rate for the road temperatures and snow intensity at the right time. Therefore, improved employee training on how and when to implement maintenance strategies and use the correct tool will improve the cost-effectiveness of winter maintenance operations.

4) More frequent and extensive application. Currently anti-icing is used more frequently for critical infrastructures, e.g. bridges, intersections and hills. In light of the excellent performance of anti-icing and pre-wetting, most respondent states are interested in increasing the frequency and serviceable range of these strategies.

CONCLUDING REMARKS

This work explored a series of improved user experience about current winter maintenance practices in North America, application status and changes in anti-icing and pre-wetting strategies over the last decade, and updated scientific understanding of these two innovative winter maintenance strategies. Together with a literature review of recent anti-icing and pre-wetting studies, a follow-up survey with a focus on performance was conducted in April 2014 to obtain the latest status on anti-icing and pre-wetting application strategies in North America. The data analysis results demonstrated the wide application and sharp increase of anti-icing and pre-wetting treatments, when compared the usage reported in a similar 2005 survey. The increased implementation and positive feedback suggested that anti-icing and pre-wetting now play an indispensable role in winter maintenance activities. Their strong performance in reducing material use, lowering maintenance cost, improving operation efficiencies, and enhancing roadway travelling conditions have gained great attention among transportation agencies. In addition, changes that have occurred related to other maintenance strategies (e.g. snow-plowing, de-icing, and sanding), total snow and ice control operation costs, technical and environmental advantages and disadvantages, practical concerns, and community response were also discussed in this work. The responses obtained from participant states and provinces may not reflect the situations of all agencies in North America, but to some extent, they reflect the current trends in winter maintenance operations worldwide.

ACKNOWLEDGEMENTS

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Crack Surveying Methods to Evaluate Sealing Practice in Alaska

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Abstract

It has long been suggested that a much more economical and sustainable approach be developed to address crack sealing effectiveness in Alaska. The development of a new method named special thermal crack evaluation (STCE) in a previous study served to directly and efficiently guide the crack sealing practices as opposed to the other nationally-used pavement surveying methods, including Long Term Pavement Performance (LTPP) and Pavement Surface and Evaluation Ratings (PASER) . However, no or little research has been found to compare and correlate STCE with the other two methods. This paper presents a study to fulfill this research need. Further data interpretation showed that STCE was capable of guiding the decision-making on crack sealing practices, while the other two methods provided quantitative measurements and general rating of the sealed or non-sealed cracks. In addition, the correlation between STCE and the other two methods recommended that STCE better be used along with the other two methods for a better and more complete guidance on current or potential crack sealing practices.

INTRODUCTION

Crack sealing is one of the most common pavement preservation techniques for countries and states (Eaton and Ashcraft 1992; Shatnawi 2008; Peshkin et al. 2011; Hicks et al. 2012). Most state transportation agencies in the U.S. have developed various regional practices. Every year in Alaska, a substantial amount of fund has been spent on sealing cracks according to practice from Department of Transportation (DOT) Maintenance and Operation. However, there is no guideline available so far to clearly instruct when a good timing is for crack sealing or under what condition cracking sealing is necessary. Therefore, there is a need to evaluate effectiveness of cracking sealing technique of asphalt concrete (AC) pavements in Alaska.

Recently, a new method named “special thermal crack evaluation” (STCE) was developed by pavement practitioners in Alaska to evaluate current sealing practices for thermal cracks, based on surveying on old AC pavements in northern and central Alaska (Mullin et al. 2014). The STCE method serves a specific purpose to efficiently guide the crack sealing practices, rather than provide typical pavement surveying data as opposed to other commonly-used methods, including Long Term Pavement Performance (LTPP) and Pavement Surface and Evaluation Rating (PASER). Upon use of STCE, recommendations for crack sealing can be made by answering several critical questions that are important to Alaska’s pavement maintenance. The answers to these questions are interpreted as ordinal data format and collected as the results of STCE. Therefore, the implementation of STCE relies on experienced field personnel that are capable of recognizing/describing all aspects of pavement surface damage and maintenance techniques, which is a similar requirement for performing LTPP and PASER evaluation.

The motivation of STCE method was to provide direct recommendation for decision-makers on crack sealing practices, and the previous study showed its successful application in Alaska (Mullin et al. 2014). However, little comparison or correlation between STCE and other commonly-used pavement surveying methods has been mentioned in the literature, which may impair the potential feasibility and applicability of STCE for pavement preservation. In order to fill this knowledge gap, this work presents a study to conduct preliminary comparison and correlation between LTPP, PASER methods and STCE.

PAVEMENT SURVEYING METHODS

STCE. The STCE data is collected in terms of answers to three basic questions that are important to Alaska’s pavement maintenance (McHattie et al. 2013). These questions include how traffic affects thermal cracking, if the interaction between thermal cracking and traffic produces additional damage to AC pavements and if sealing of thermal cracks improves the pavement performance.

In order to simplify the data collection process, thermal cracks are characterized into two groups: major and lesser (Figure 1, Mullin et al. 2014). As can be seen in Figure 1 (a), the major transverse cracks are observed to extend or mostly extend across the entire width of the pavement, and tend to be oriented perpendicular to the roadway centerline. Figure 1 (b) displays typical lesser cracks characterized in STCE that are interconnected in a map-like or blocky grid. Therefore, the aforementioned three basic questions can be expanded to eight specific questions displayed in Table 1, to which the answers can be recorded in an ordinal format.

(a) Major transverse thermal cracks. (b) Lesser transverse thermal cracks.
Figure 1. Typical major and lesser cracks characterized in STCE.

Table 1. Eight questions developed for STCE method.

Question	Description
A	Condition of major transverse cracks comparing at the wheel path versus non wheel path (1 = no difference, 2 = slight difference, 3 = much difference).
B	Condition of lesser thermal cracks comparing at the wheel path versus non wheel path (1 = no difference, 2 = slight difference, 3 = much difference);
C	Maximum observed width of major transverse crack zone (1= <2 inch, 2 = 2 to 5 inch, 3 = > 5 inch).
D	Maximum observed width of lesser thermal crack zone (1 = <1/8 inch, 2 = ≥1/8 inch).
E	General pavement deterioration due to major transverse cracking (1 = none, 2 = slightly noticeable, 3 = very noticeable).
F	General pavement deterioration due to lesser thermal cracking (1 = none, 2 = slightly noticeable, 3 = very noticeable).
G	Presence of crack sealant (1 = no/very old sealant, 2 = majors sealed, 3 = majors + lessers sealed).
H	Present condition of sealant (1 = old and/or re-cracked, 2 = some re-cracking, 3 = mostly good condition, 4 = no sealant).

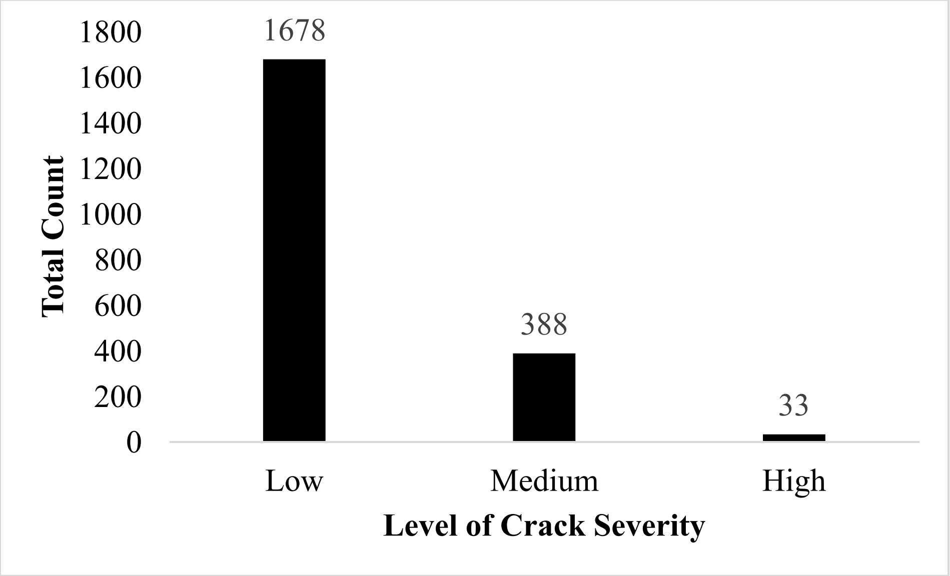
The STCE data used for analysis in this study was collected from a sample size of 91 locations on old pavements in Northern and Central Regions of Alaska, which was consistent with the previous study (Mullin et al. 2014). All sample locations were 161 meter (0.1 mile) in length.

LTPP. The Long Term Pavement Performance Program (LTPP) was initiated as a part of the Strategic Highway Research Program (SHRP) (Miller and Bellinger 2003) and continued under the Federal Highway Administration (FHWA) when SHRP ended in 1992. LTPP was developed with the purpose to provide information consisted of climate, traffic volumes and loads, and pavement surface condition, for designing improved and longer-lasting pavements. The data are recorded in two forms: quantitative measured values in the total surveyed section and mapping distresses in 15.25-meter (50-foot) increments. Distresses in LTPP are identified based on the FHWA manual "Distress Identification Manual for the Long Term Pavement Performance Program" (Miller and Bellinger 2003), including five major categories: cracking, patching and potholes, surface deformation, surface defects and miscellaneous defects. Typically, a section of 152.4-meter (500-foot) is surveyed as one location for LTPP. For this study, however, the same section of 161-meter (0.1 mile) in length was surveyed to keep the consistence with the aforementioned STCE surveys.

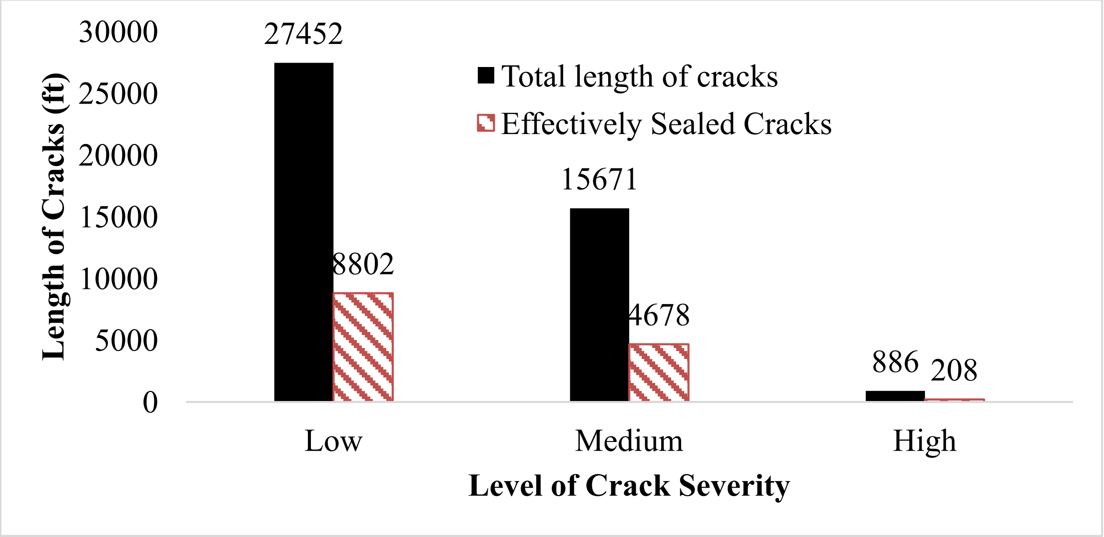
PASER. PASER was developed by the Wisconsin Transportation Information Center as a road surface condition rating system (Walker 2002). The PASER results range from 1 to 10, which combine severity level of various road surface conditions into a single number. The rating of 1 stands for a totally failed pavement while the rating of 10 indicates a newly constructed roadway. Four major categories of distresses that are evaluated in PASER include surface defects, surface deformations, patches and potholes, and cracks. Different from quantitative values recorded in LTPP, the severity of each distress in PASER is described as follows: n – none, l – low, m – medium, and s – severe, which makes PASER a quick-running survey of roadways that can be accomplished by visual assessment. For this study, PASER data were collected in the same locations with the other two surveying methods.

COMPARISON IN DATA INTERPRETATION OF EACH METHOD

LTPP and PASER. The interpretation of LTPP and PASER data selected in this study focused only on transverse (thermal) cracking measurements for comparison with STCE data. For LTPP surveying of transverse cracks, three categories were taken into account including total count of cracks, total linear length of cracks and total linear length of effectively sealed cracks. Each category was subdivided into three levels of crack severity: low, medium and high. Figure 2 presents a typical interpretation of LTPP data related to transverse cracks. The significant observation in Figure 2 (a) can be that very limited amount of high-severity transverse cracks were observed in the surveyed sections (a total of 33 cracks in 91 sections). It can be seen in Figure 2 (b) that less than 1/3 of the total lengths of low- and medium-severity transverse cracks, and less than 1/4 of the high-severity cracks were effectively sealed. These observation and information revealed the current conditions of thermal cracks and crack sealing practices in the selected locations, however, no conclusion can be made on how the sealing practice helped to improve the pavement performance.



(a) Count of total thermal cracks.



(b) Length (ft) of thermal cracks.

Figure 2. Typical data interpretation of LTPP method (Note: 1 ft = 0.3 m).

Figure 3 displays a typical PASAR results based on surveys of transverse cracks in this study. Only data collected from 90 sites are presented herein due to missing record in one location. The significant observations are: only 1 out of 90 locations showed no cracking; transverse cracks in 92% (83 out of 90) of selected sites were rated low or medium levels of severity; 6 out of 90 sites exhibited severe transverse cracks. These data simply indicated a general view of the rating or severity level of current thermal cracking condition in the surveyed pavements. No recommendation can be made on the crack sealing practices.

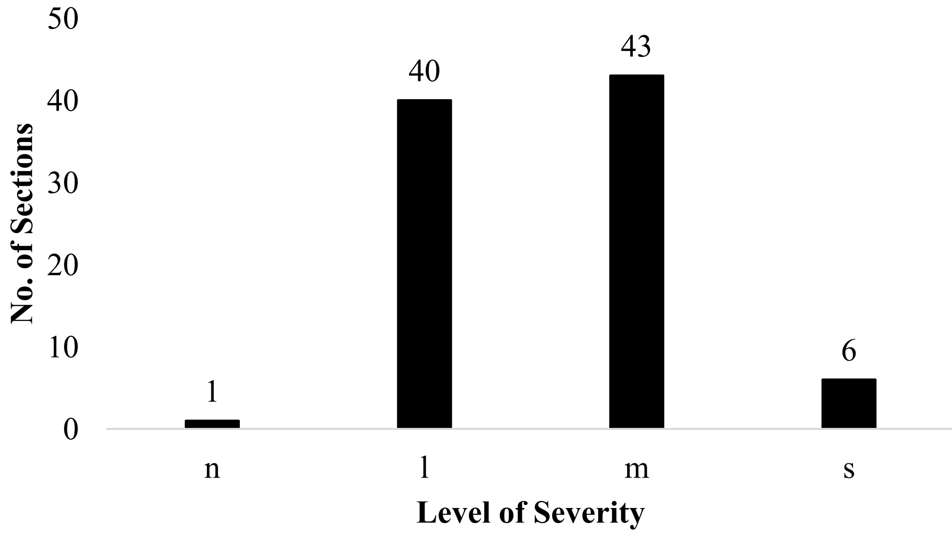
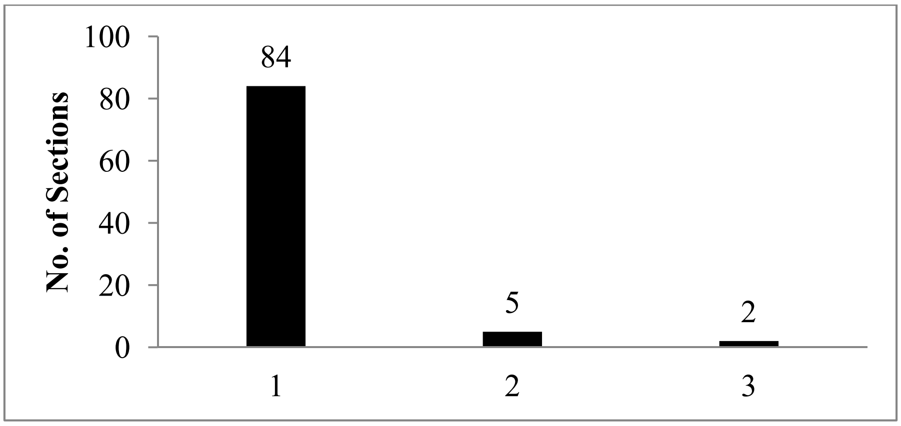
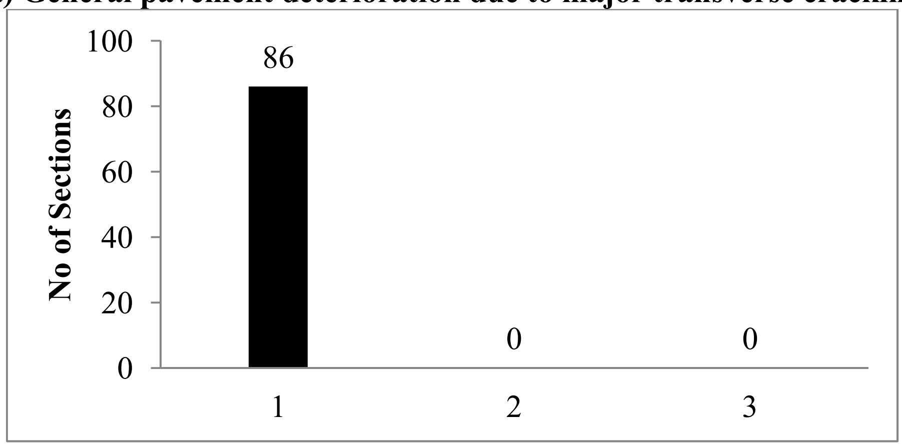


Figure 3. Typical data interpretation of PASER method (Note: n – none, l – low, m – medium, and s – severe).

STCE. The STCE data were derived from responses to the eight questions listed in Table 1. One frequency histogram can be built for each question. Since the results of STCE method were reported in the previous study (Mullin et al. 2014), responses to questions E and F in Table 1 are presented herein as typical results of STCE surveying, which can be seen in Figure 4. The interpretation of Figure 4 can be straightforward: 8% (7 out of 91) of the surveyed sites were found to have major cracks contributing to pavement deterioration, while none of the sites showed signs of lesser transverse cracks affecting the pavement performance. As a result, a general recommendation was made in the previous study (Mullin et al. 2014) that lesser cracks be sealed while the major cracks not be sealed. Responses to other questions in Table 1 may lead to other recommendations on crack sealing practices, however, are not discussed in this part due to paper length.



(a) General pavement deterioration due to major transverse cracking.



(b) General pavement deterioration due to lesser transverse cracking.
Figure 4. Typical data interpretation of STCE method (1 = no difference, 2 = slight difference, 3 = much difference).

CORRELATION OF METHODS

As discussed above, LTPP, PASER and STCE are developed with different purposes and focuses. However, the information revealed by each surveying method may be correlated or complemented by one another, which may lead to more reasonable conclusions or recommendations. This motivated the following research based on a preliminary statistical analysis.

LTPP vs. STCE. Responses to questions E and A from STCE data were used as examples to correlate with the LTPP transverse cracking data in this study. Table 2 presents the arranged results by combining responses to STCE question E and LTPP data. In order for potential correlation with STCE, each crack recorded in LTPP was marked as either lesser or major cracks by the surveyor. The table itself does not impart useful information rather than quantified total and sealed lengths of major transverse cracks. A better analysis can be accomplished by calculating the major crack sealing ratio conditioned on each category of STCE responses, which is displayed as histogram in Figure 5. The major crack sealing ratio can be calculated as the ratio of sealed length of major cracks over total major crack length displayed in

Table 2. It can be seen from Figure 5 that a lowest average major crack sealing ratio was found on locations where major transverse cracks were not considered as a significant factor of pavement deterioration. This finding may contradict to the common sense that more sealants placed on the major cracks will result in less pavement deterioration due to major transverse cracks. A re-assessment of necessity of crack sealing on major cracks is highly recommended based on this finding. However, it should be noted that only 5 and 2 sites were characterized as “slightly noticeable” and “very noticeable” for STCE question E, respectively. The small sample size may have affected the analysis.

Table 2. STCE data of question E vs. LTPP transverse cracking data.

Responses to STCE question E	1	2	3
No. of sections	84	5	2
LTPP total length of major transverse cracks (ft)	6607	276	163
LTPP sealed length of major transverse cracks (ft)	2126	118	96

Note:
STCE questions E - General pavement deterioration due to major transverse cracking (1 = none, 2 = slightly noticeable, 3 = very noticeable).
1 ft = 0.3 m.

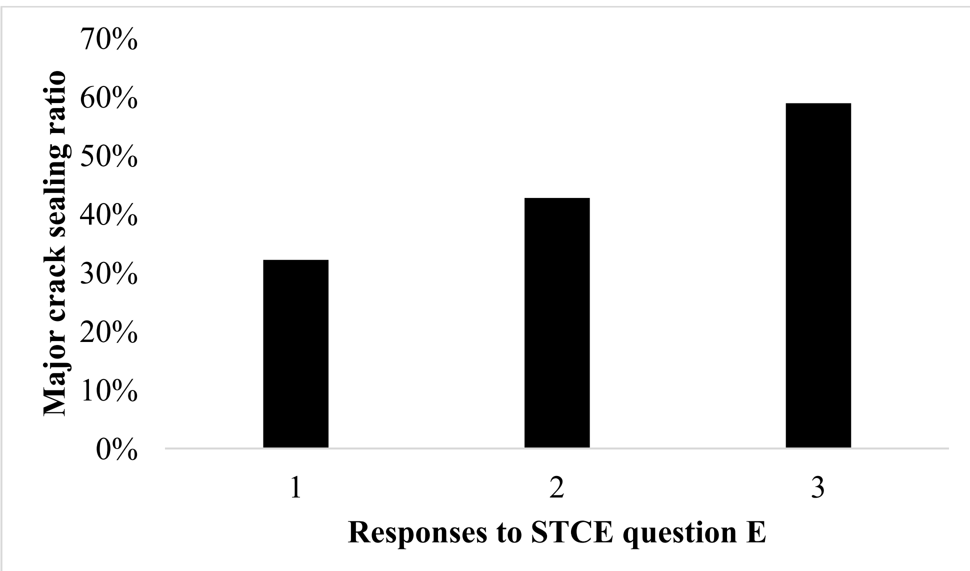


Figure 5. Major crack sealing ratio conditioned on STCE question E.

Similar correlation can be conducted between STCE data of question A and LTPP transverse cracking data. Table 3 presents the combined data while Figure 6 shows the results of major crack sealing ratio. The two categories of STCE responses “slight difference” and “much difference” were added up due to small size of samples. According to Figure 6, the major crack sealing ratio was found to be much higher at STCE response “no difference” than at “slight difference” and “much difference”. This indicates that sealing on major transverse cracks may improve the condition of the cracks and make it more consistent at either the wheel path or non-wheel path.

Table 3. Combination of STCE data of question A and LTPP transverse cracking data.

Responses to STCE question A	1	2 & 3
No. of sections	59	31
LTPP total length of major transverse cracks (ft)	4664	2337
LTPP sealed length of major transverse cracks (ft)	2118	443

Note:
STCE questions E - Condition of major transverse cracks comparing at the wheel path versus non wheel path (1 = no difference, 2 = slight difference, 3 = much difference).
1 ft = 0.3 m.

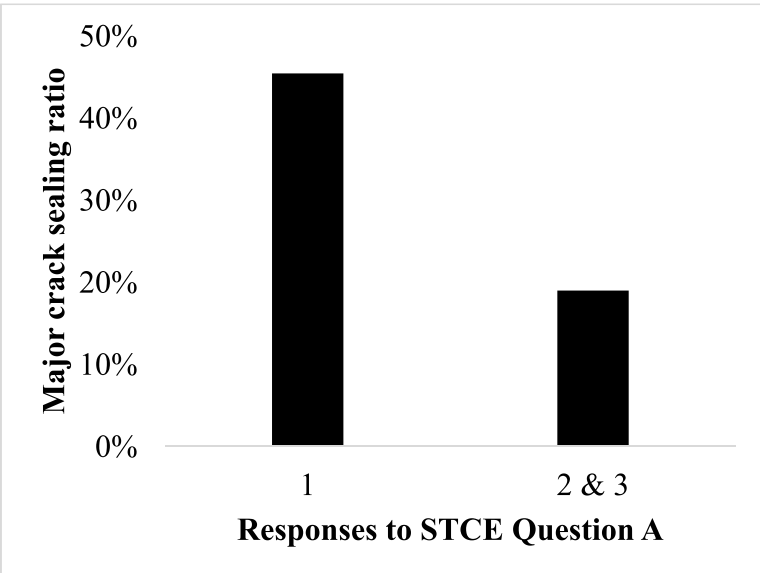


Figure 6. Major crack sealing ratio conditioned on STCE question A.

The findings drawn from Figures 5 and 6 appear to contradict to each other, since the former questions the effectiveness of sealing major cracks while the latter supports the same type of crack sealing. This is caused by the different evaluating aspects of each STCE question. Recommendation based on Figure 5 focused on the effects of sealing major transverse cracks in terms of the general pavement performance, while the one made by Figure 6 highlighted the differences of pavement conditions at wheel or non-wheel paths. Actually, a combination of these findings may lead to a more complete conclusion. It can be interpreted as: sealing major transverse cracks upgrades crack conditions but may not improve the general pavement performance. Findings like this may help to make better decision on crack sealing practice, and cannot be obtained by a single method mentioned above.

PASER vs. STCE. Different from LTPP, PASER gives a general severity level of each distress: n – none, l – low, m – medium, and s – severe. Tables 4 and 5 organize the combined PASER cracking data and responses to the same two STCE questions used in “LTPP vs. STCE” section, respectively, in terms of PASER severity frequencies conditioned on each STCE question.

Table 4. PASER severity frequencies conditioned on STCE question E.

PASER cracking	Responses to STCE question E		
	1	**2**	**3**
n	1	0	0
l	31	7	2
m	26	10	7
s	1	4	1

Table 5. PASER severity frequencies conditioned on STCE question A.

PASER cracking	Responses to STCE question A		
	1	**2**	**3**
n	1	0	0
l	39	1	1
m	38	4	1
s	6	0	0

Similarly, a better analysis can be achieved by calculating the frequency ratio of each severity level. Due to limited size of sample, lower levels of crack severity was used to represent the summation of “n” (“none”) and “l” (low”) levels, while the STCE response “2” (“slight difference”) and “3” (“much difference”) are added up. Therefore, the ratio of lower levels of crack severity conditioned on response “1” of STCE question E can be calculated as: (1+31) / (1+31+26+1) * 100% = 54%.

Figure 7 presents the ratios of lower levels of PASER crack severity conditioned on STCE questions E and A. It can be seen that both ratios decreased significantly when the response changed from “1” to “2 & 3” regardless of the STCE question. According to the definitions of questions E and A in Table 1, this finding indicates that: a higher possibility of cracks falling on lower severity levels may occur, if in a STCE survey general pavement deterioration is found not to be related to major transverse cracking, or if condition of major transverse cracks comparing at the wheel path versus non wheel path is found to be no difference. This finding may not directly help to make decisions on crack sealing practices. However, it may strengthen and complement the STCE survey results.

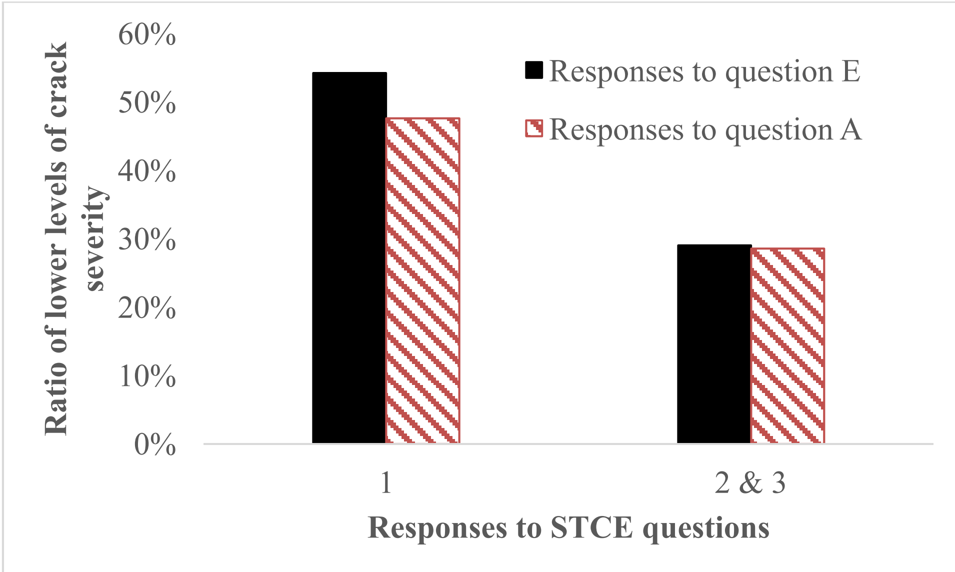


Figure 7. Ratio of lower levels of PASER crack severity conditioned on STCE questions E and A.

SUMMARY AND CONCLUSIONS

Since the mid-1980s, it has been strongly suggested that a much more economical and sustainable approach be developed to address the thermal cracking issues in Alaska (McHattie et al. 2013). The development of STCE method has shed light on a new direction of surveying and evaluating present thermal cracks, rather than simply followed the traditional "see a crack – seal a crack" approach (Mullin et al. 2014). This paper further compared STCE to the other two widely-used pavement surveying methods, LTPP and PASER. It was found that LTPP served to reveal the current conditions of sealed or non-sealed thermal cracks, and PASER simply provided an overview of the severity level of the current cracking condition. On the contrary, STCE surveying functioned to directly help to guide the decision making on current or potential crack sealing practices.

In addition, this paper conducted correlation between LTPP, PASER and STCE methods. It was found that LTPP and PASER would work well with STCE although analyses of a few selected examples were presented only. The combination of LTPP and STCE results led to a more complete conclusion that could not be obtained by any single method, while PASER data helped to strengthen and complement STCE, which makes STCE results more convincing and valid. Therefore, it is highly recommended that STCE along with the other two methods continue to be used to survey more locations in order for better analysis of crack sealing effectiveness.

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Sustainable Construction in Remote Cold Regions: Methods and Knowledge Transfer

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Abstract

This project aims to identify sustainable construction techniques appropriate for remote and cold regions, noting extension to operations and maintenance as applicable. This paper explains the methods used to review the vast body of literature regarding green construction in warm regions, and the use of a hierarchal taxonomy to categorize the information found in order to reduce it to a form useful for eventual presentation to engineering and construction managers.

INTRODUCTION

Remoteness and cold climate make construction in rural Alaska challenging and expensive. Sustainable construction practices serve to reduce the negative environmental consequences of construction projects and often reduce their life-cycle cost. While such practices are commonly considered in warm climate construction, especially for vertical construction, little knowledge has been accumulated about sustainable construction in cold regions. Besides the constructed work, remote projects often require their own infrastructure, such as camps and supply depots. In addition, most construction in remote cold regions is horizontal construction, requiring continuing maintenance and operations (M&O) efforts, likewise with its own infrastructure. Since most M&O for these kinds of projects uses the same types of equipment and infrastructure, we include this work under the rubric of "construction." Although we focus on the field construction, typically by a contractor as opposed to the owner's own forces, there is often little to distinguish those with respect to the operations. Likewise, we cannot exclude design, insofar as the specifications may require certain features during the construction, and/or include permit stipulations. Finally, in a design-build or CMAR project delivery, the contractor may include sustainable features as an enhancement in the design competition. Here we present our process for winnowing down the vast amount of "green construction" items in the literature to those possibly relevant to remote cold regions, and then our plan to review these with remote cold regions experts and revise and add new items based on their expertise.

BACKGROUND

Many definitions of sustainability have been suggested. All revolve around the importance of creating and maintaining conditions under which human and natural needs can be met both in the present and in the future. Thus, “a sustainable society is one that can persist over generations, one that is far-seeing enough, flexible enough, and wise enough not to undermine either its physical or its social systems of support” (Meadows et al, 1992). An enduring classic definition of sustainability was suggested by the World Commission on Environment and Development as follows: “development that meets the needs of the present without compromising the ability of future generations to meet their own needs” (quoted in Burrow et al, 2013).

As noted by Burrow (2013), it is common to represent sustainability in three dimensions, including the economy, the environment and society. From this point of view, sustainability is achieved only when there is a balance among those three aspects, implying the need, often, for tradeoffs and compromise among the three. In the context of the study reported in this paper, a sustainable construction project would be one for which there is a balance among present and future needs for economic viability, a flourishing society, and a healthy natural world. For this project, and in the spirit of the theme of this symposium, we confine our consideration to environmental sustainability while recognizing the importance of the other two aspects of the overall sustainability challenge.

It follows from the above discussion of environmental sustainability that, within the context of the construction process, sustainability involves “creating construction items using best-practice clean and resource-efficient techniques from the extraction of the raw materials to the demolition and disposal of its components.” (Yates, 2014) The project described in this paper examines one relatively small but significant aspect of sustainable construction – environmentally responsible methods for conducting field construction and maintenance operations in remote cold regions.

Sustainable, or “green,” construction has been an essential part of the building construction process for many decades. The contemporary interest in such practices arose in the 1970’s when rapid increases in world oil prices instigated improvements in building energy efficiency. Furthermore, the environmental movement that began in the 1960’s and 70’s spurred the development of environmentally friendly and energy efficient buildings and other structures (EPA 2014).

A review of the literature of sustainable construction (see, for example Clemente 2007, U.S. Green Building Council 2014, Santos et al 2007, Institute for Sustainable Infrastructure 2014) reveals two important realities about the work accomplished to date. First, “sustainable construction” embraces all phases of the project life cycle, from early planning to operations, maintenance and even decommissioning, but the greatest emphasis has been on the design and material specification aspects of the project development process. Second, the emphasis to date has been on buildings and other such structures (“vertical” construction), with much less consideration of such practices in heavy construction (“horizontal” construction), such as roadways, pipelines, boardwalks, and airfields. For example, a typical Internet entry explaining sustainable, or “green,” construction (GSC 2014) states, in part, “Green buildings are designed to reduce the overall impact of the built environment on human health and the natural environment by: 1. efficiently using energy, water and other resources, 2. protecting occupant health and improving employee productivity, 3. reducing waste, pollution and environmental degradation." Note the emphasis on buildings and design.

There is a need, especially in remote cold regions such as rural Alaska, for a compilation of guidelines to assist construction contractors in conducting their operations in environmentally responsible ways. Since a large proportion of construction efforts in such areas develop horizontal projects, it is for this kind of work that such guidelines will be most helpful. The purpose of this study, then, is to identify and codify practical environmentally sustainable construction practices for use by contractors operating in remote cold regions and to convey the findings in useful form to those who can use them.

This paper is a status report on an on-going investigation. It describes the work accomplished to date and sets forth planned tasks to be carried out by project completion in late 2015.

PROJECT OBJECTIVES AND WORK PLAN

As set forth in the proposal for the project (Perkins 2014), the primary objectives are as follows:

1. Translate common sustainable construction methods in use for vertical construction into similar approaches for horizontal construction for use as a checklist of topics for horizontal construction in remote and cold regions.
2. Identify appropriate sustainable horizontal construction methods for use in remote regions and in severe climates
3. Develop guidelines and means for the formal transfer of such methodologies to those who conduct construction operations in such environments. Examine practicality of such techniques for use in remote and harsh environments by rural residents.

The project plan includes the following tasks:

1. Compile inventory of sustainable construction methods currently used in vertical construction, from existing compendiums, sustainability organizations, and worldwide literature and meetings with Alaskan experts.
2. Compile inventory of sustainable construction methods currently used in horizontal construction, from existing compendiums, sustainability organizations, worldwide literature and meetings with Alaskan experts.
3. Identify current construction practices in rural Alaska that have, or could have, a sustainability component, noting particular challenges due to a) cold climate aspects and b) rural aspects. Draft a preliminary report.
4. Organize and conduct a gathering of construction industry leaders, in either a short course or seminar setting, to review findings to date and seek guidance as to potential changes in practices that could lead to greater sustainability.
5. Revise preliminary report and develop guidelines for selected practices.

6. Design an outreach course for use with rural Alaskan construction personnel that sets forth suggested practical sustainable construction methodologies - a manual, teaching materials or multi-media presentation.

7. Revise existing UAF construction course(s) to incorporate sustainability practices for rural Alaska, and/or develop a new course that emphasizes horizontal construction.

To date, the investigation has proceeded through task 3. This paper serves as the preliminary report called out in Task 3. The following section summarizes the literature search that identified various construction-caused environmental impacts and potential ways to deal with such impacts.

REVIEW OF SUSTAINABLE CONSTRUCTION IMPACTS AND PRACTICES

As noted above, much of the emphasis on sustainable "construction" to date has been on the design phase of the construction project life cycle, with significant developments in material design and specification and the development of systems to make completed projects more environmentally efficient. Also, such developments have tended to apply more to vertical, rather than horizontal, construction. However, a review of the literature reveals that there is a substantial body of information related directly to the field construction process itself. Furthermore, it is clear that many of the environmentally sustainable methods used by contractors in building construction can apply equally to the construction of roadways, airfields, and other horizontal facilities.

The United States government, through its Environmental Protection Agency, enforces federal mandates that must be followed by all industries, including construction, in complying with a large number of environmental laws and regulations. A manual, *Managing Your Environmental Responsibilities: A Planning Guide for Construction and Development* (U.S. Environmental Protection Agency 2005) is available. A guide that interprets this manual has been published for contractors (U.S. Environmental Protection Agency n.d.).

To assist its members and other contractors in interpreting and meeting federal and other environmental requirements, the Associated General Contractors of America provides guidance on developing company- or project-specific environmental management systems (Associated General Contractors 2004a and 2004b).

Two books about sustainable, or green, construction (Glavinich 2008; Kibert 2008) provide helpful guidance throughout the project development life cycle; each contains sections specific to field construction operations. Likewise, the Institute for Sustainable Infrastructure (2014) provides a system for evaluating various aspects of a project's development, including field construction, through its Envision™ system. The U.S. Green Building Council (2014) sponsors the well-known LEED (Leadership in Energy and Environmental Design) rating system. The system is confined to buildings and is mostly about design, but some of the concepts apply to building construction operations. In an attempt to apply LEED's building-oriented concepts to road building, a system called Greenroads™ (Muench et al 2011) was developed.

This rating system covers both design and construction of roadways; construction activities include site recycling planning, fossil fuel reduction, equipment emissions reduction, paving emissions reduction, and water tracking.

The most relevant and complete document related to our study appears to be the *Field Guide for Sustainable Construction* (Pulaski (2004). Ten chapters, each covering such topics as site/environment, waste prevention, recycling and energy, include sections organized roughly by construction specification (sitework, foundations, substructure, and the like). Each has specific suggestions for environmentally effective contractor actions, plus case studies and references. In addition, a paper by Pearce et al (2010) reports a benchmark study of best sustainability practices in construction; the study utilized interviews and observations to "better understand what types of sustainability-related innovations are most easily and effectively adopted over time by project teams on capital projects."

With regard to specific construction-related environmental impacts and means for dealing with them, the matter of construction waste and demolition debris is of major concern in many places in the world (Couto et al 2007, Liao et al 2011, State of Washington Department of Ecology 2014, Tamraz et al 2011, Zhang et al 2011). Air pollution, especially dust caused by equipment operation on roadways and the construction site, has been dealt with by several authors (Anderson 2014, Barnes et al 2014, James 2014, Skorseth et al 2000, Withycombe et al 2006); this problem is of special concern in rural Alaska.

Water pollution of many kinds is a major concern on most construction sites. In the United States, there are stringent requirements for storm water pollution prevention, and contractors have become very familiar with contractual requirements for a SWPPP (Storm Water Pollution Prevention Plan) and have learned to comply or face substantial penalties (U.S. Environmental Protection Agency 2005). A presentation Nowack (2014) contains helpful guidelines for the prevention of many types of water and other pollution.

Among other construction site environmental issues in the literature, there is concern about energy efficiency on construction sites, including the generation and delivery of electric power and the efficient use of power for tools, lighting, and signage (Sharrard et al 2007).

A TAXONOMY

We next draw from the literature cited above, other literature sources, our own personal experience (a combined 90 years in construction and over 80 years in Alaska for the two authors) and that of our colleagues to compile as large an array of potential negative environmental effects as we could find. As a starting point, we include any and all, without regard to the type of construction or whether it is remote or more urban, cold climate or more temperate. Ultimately, as suggested earlier in the paper, we intend to focus on impacts that can occur in remote cold regions and methods for dealing with those impacts. So as to lower the chance of neglecting some of those, our research method is to start with as many as possible and refine as the study proceeds.

One of the project's biggest challenges so far has been how to organize those findings into a reasonable and useful outline or taxonomy. Many writers (Huang 2011, for example) start by listing the various kinds of environmental impacts – water pollution, solid waste production and disposal, and the like – as major categories and then discuss the operations that produce such impacts and then some potential means for dealing with those impacts. We decided, instead, to begin the outline from the contractor's viewpoint by listing, as major categories, the construction operations or activities that can potentially produce environmental impacts, then identify those impacts and, finally, suggest possible methods for avoiding, reducing and/or mitigating those effects. Thus, in Figure 1, we show a generalized construction project activity taxonomy, broken down into the major categories of project support, the project itself, and operations and maintenance.

Within each Major Category, we list type of work activity (Worker Housing, Life Support, Office & Shops; Support Equipment Operation; and the like), and then we identify those potential negative environmental impacts associated with that work activity and attach possible measures the contractor can use to avoid, reduce and/or mitigate those impacts. Two examples are shown. In Figure 2, we show a diagram for the Support Equipment Operation work activity, while in Figure 3, we show a similar breakdown for the Site Development work activity. Note that, for Support Equipment Operations, the potential negative impacts include air and noise pollution, excessive vibration, improper fuel storage, handling and use, and pollution that can result from cleaning and storage of vehicles and equipment. For each, we show some measures that might be used to combat such impacts. For example, to respond to air pollution, the contractor might provide respiratory protection for workers, minimize equipment run time, use alternative fuel vehicles, and conduct regular inspection and maintenance activities, among others.

For Site Development, Figure 3 includes degradation of permafrost (permanently frozen ground), improper and excessive clearing, insufficient project site layout, improper handling and storage of removed soil, survey damage, improper storage tank location and operation, improper storm water management, historical and archeological sites issues, and endangered and subsistence species issues as potential negative impacts. To combat the effects of improper and excessive clearing, for example, the contractor might grind and chip trees to provide mulch, resell trees and other plants, avoid damage to existing improvements, and limit the cleared area to an absolute minimum.

At this point in the process, we have not considered whether an impact might have to be dealt with in a remote cold area, nor have we considered whether a method of contending with the impact might be practical for such a region. We deal in a preliminary way with that aspect of the study in the next section.

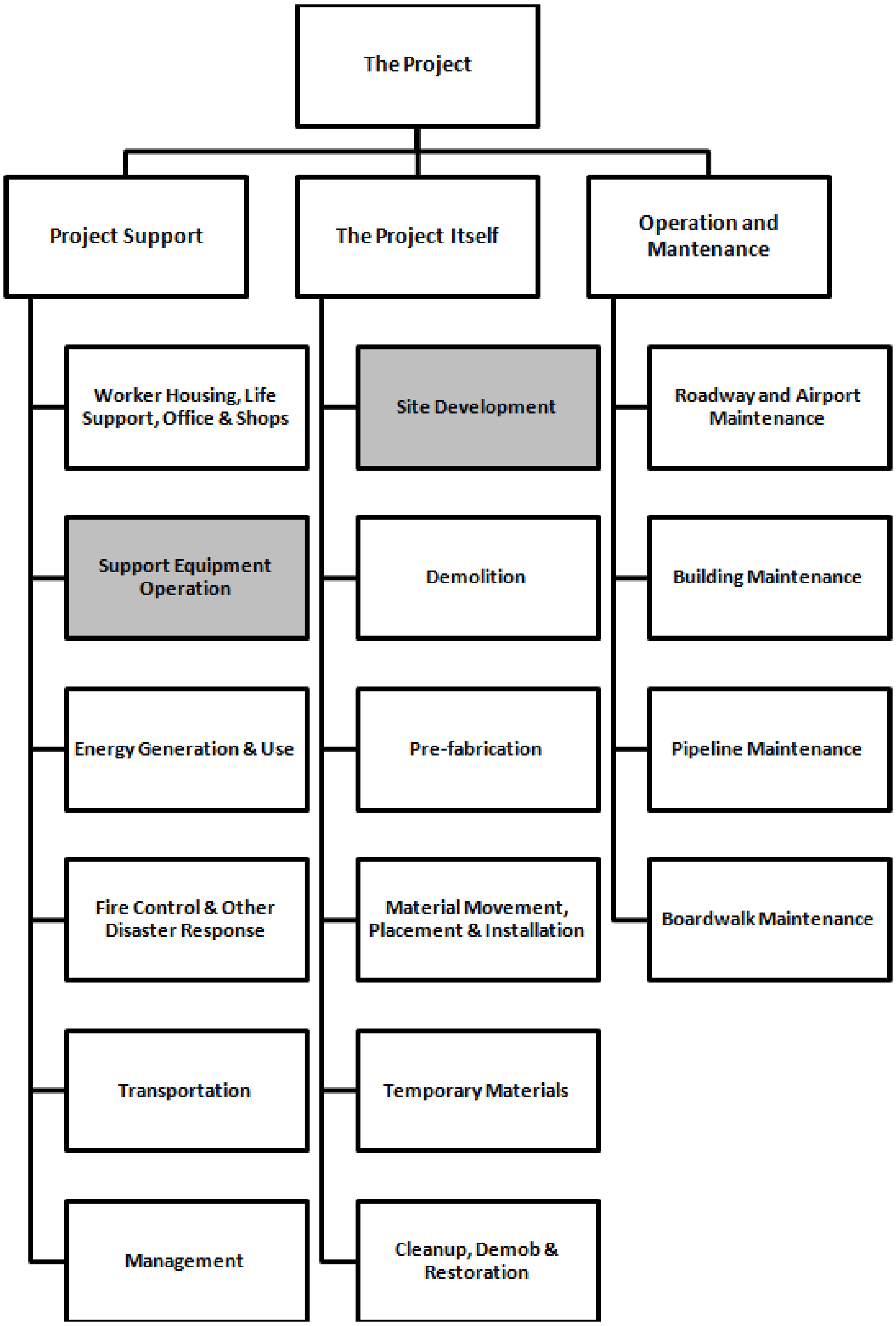

Figure 1. A construction project taxonomy (highlighted work activities are shown in detail in figures 2 & 3).

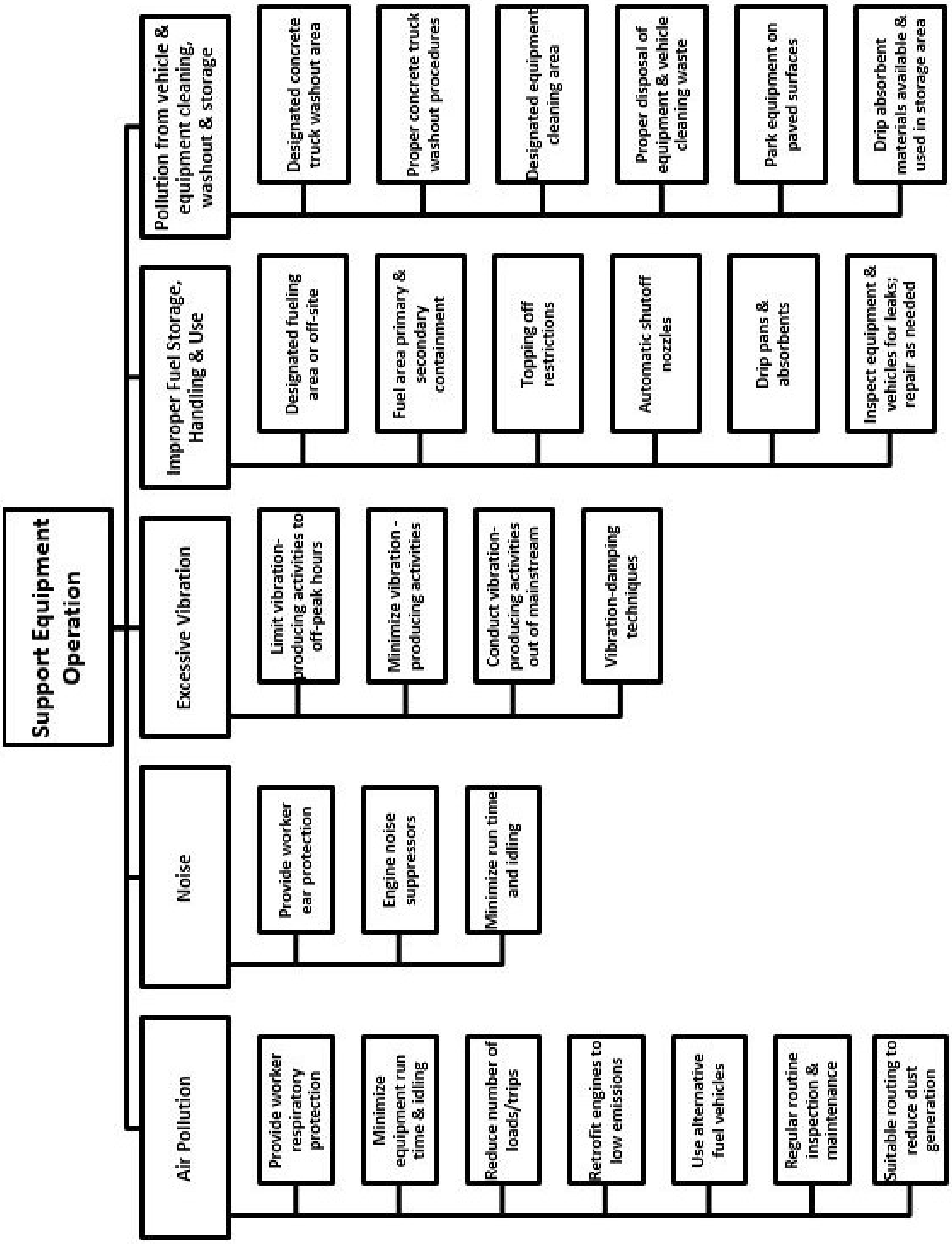

Figure 2. Taxonomy of support equipment operation impacts and mitigation methods.

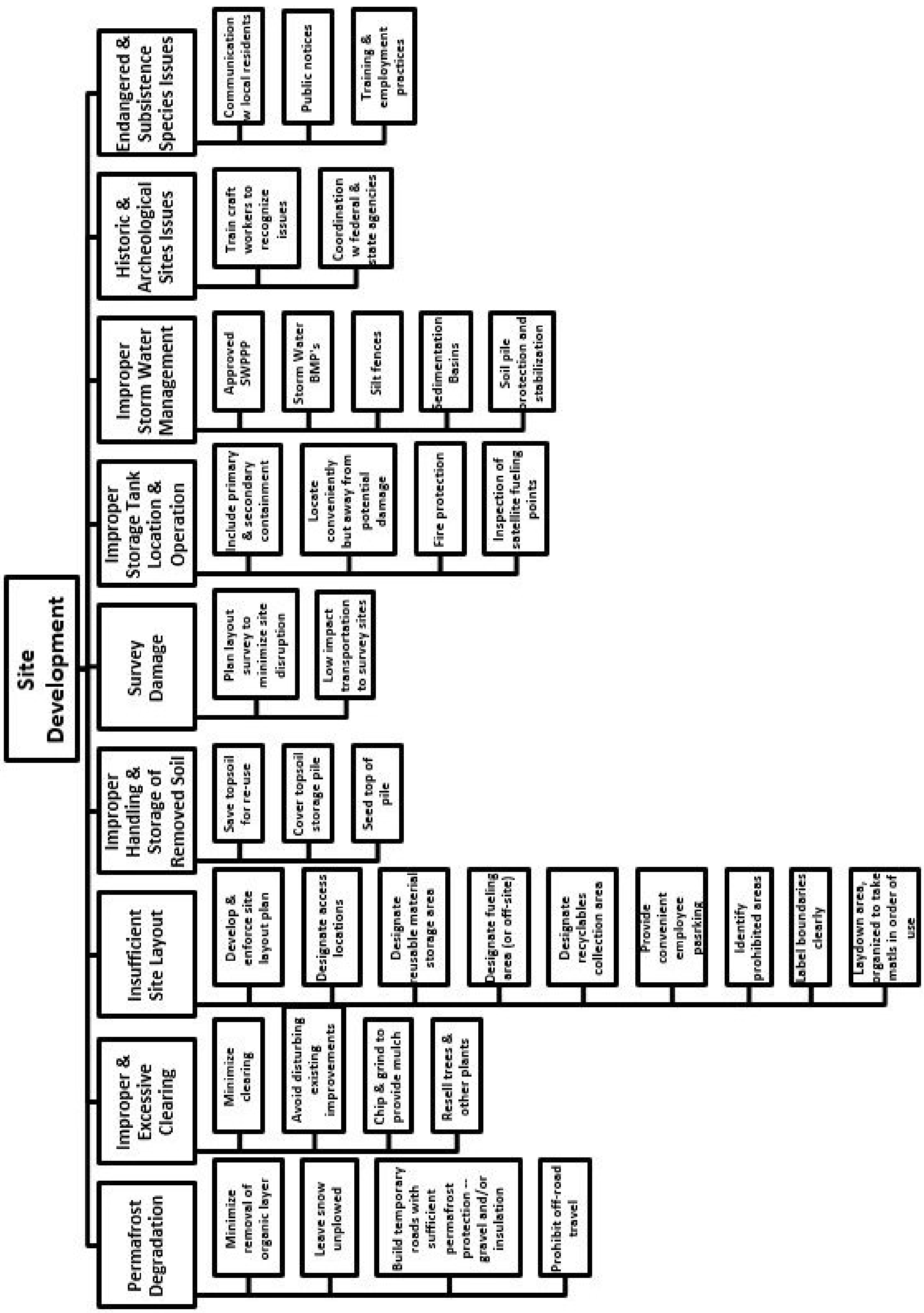

Figure 3. Taxonomy of site development impacts and mitigation methods.

PRACTICAL APPROACHES FOR REMOTE COLD REGIONS

From among the myriad of possible methods for dealing with the negative environmental impacts of field construction operations, we are currently (January 2015) in the process of identifying practical methods that could potentially avoid, reduce and/or mitigate such impacts on projects in remote cold regions such as rural Alaska. As an example, we show in Figure 4 some highlighted measures that appear to be practical for environmental impacts resulting from the Support Equipment Operation work activity. Note that these measures include, among others, providing respiratory protection for workers and minimizing equipment run time and idling to combat air pollution, designating a fueling area with primary and secondary containment and providing drip pans, absorbents and automatic shutoff nozzles to mitigate damage due to improper fuel storage and handling, and reducing pollution caused by vehicle and equipment washing and storage by using a designated concrete truck washout area and having drip absorbent materials available in equipment and vehicle parking areas. Note that a measure such as using alternative fuel vehicles is thought not to be practical because such fuels are typically not available in remote cold regions.

The completion of this identification of practical approaches, and the advice of construction experts in assisting the refinement of that list, will be essential steps in the successful conclusion of this project.

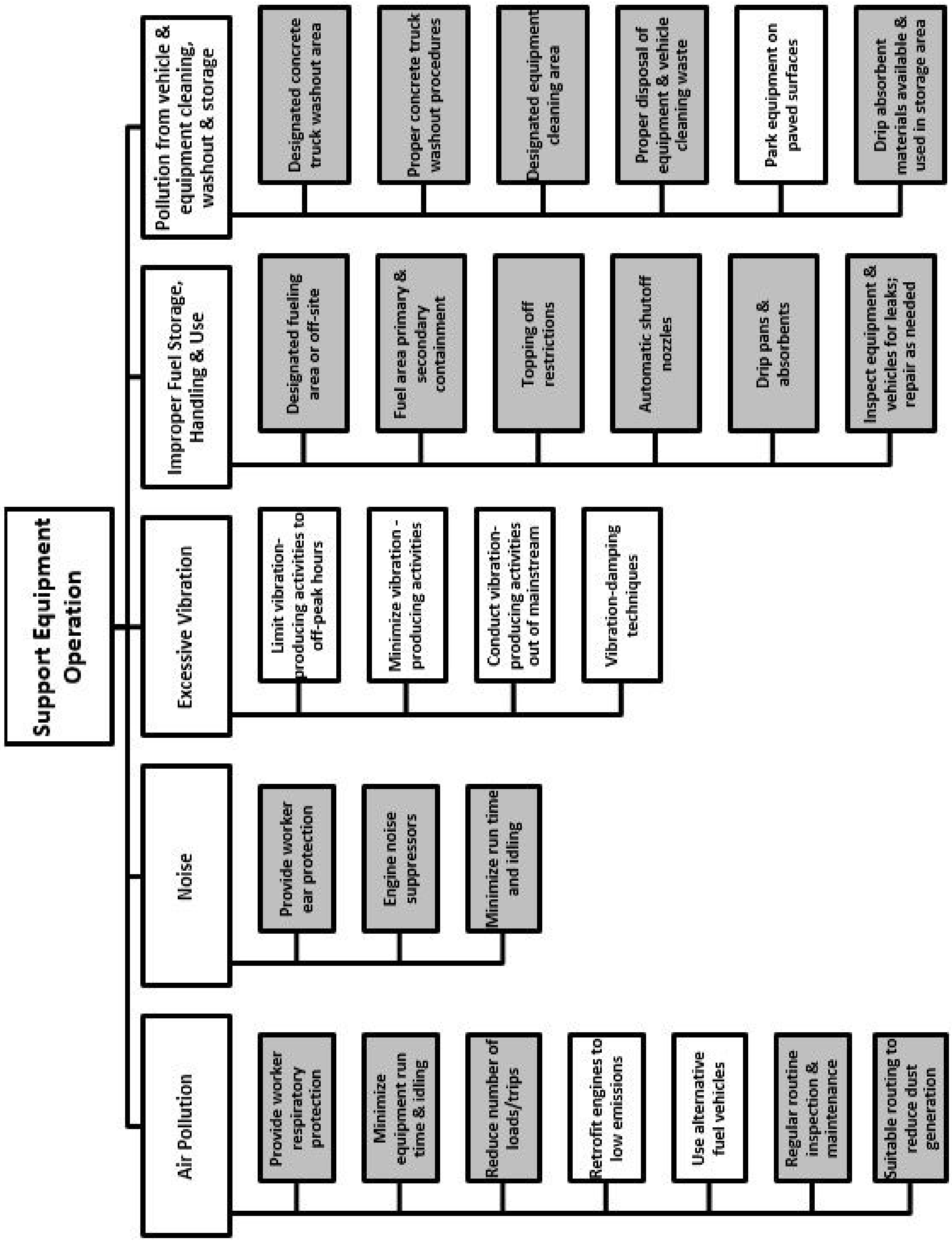

Figure 4. Taxonomy of support equipment operation impacts and mitigation methods, highlighting potential remote cold regions mitigation measures.

FUTURE PROJECT TASKS

Since this is a status report for a project whose completion is scheduled for several months in the future, it is appropriate to describe briefly the project's remaining work effort. We list those tasks below, in roughly the order we anticipate completing them.

We will fully develop a large spreadsheet (not shown in this paper) that categorizes potential environmental impacts and associated possible mitigation measures for all of the work activities shown in Figure 1. This information will be converted into diagrams similar to those in Figures 2 and 3 for each work activity, for ease of use in the tasks that follow.

On each if these detailed diagram, we will indicate our judgment as to which mitigation measures might be practical for remote cold regions, similar to Figure 4.
We will then meet with construction experts – contractors, designers, owners, facility maintenance personnel – with knowledge of and experience in construction and facility maintenance operations in remote cold regions. At these meetings, we will explain the purpose of our project and ask for their advice on the categories in our taxonomy, which items should be deleted or modified, and what might be added. We will then have them help us to refine and add to those mitigation measures they consider necessary and practical for remote cold regions. Lastly, we will ask for suggestions about how our eventual findings might best be transferred to those operating in such regions.

After digesting and organizing the results of our meetings with construction experts, we will publish a set of guidelines. The intent is to make such a publication practical and easy to use. It is expected that the guidelines will be published for distribution in hard copy and also made available electronically via the Internet.
We will conduct a meeting or series of meetings with contractors, owners and others to report our findings, distribute the guidelines and explain each of the recommended mitigation measures.

Finally, we will develop an outline of a suggested syllabus for a module on sustainable construction practices, based on our findings, which could be made a part of a standard university course on construction engineering and management. Such a module might occupy, say, a two or three hour lecture/discussion.

CONCLUSION

The body of literature regarding green construction is vast indeed. Two tasks in distilling information useful for sustainable construction in cold and remote regions are, first, to screen out the information that pertains mainly to buildings in warmer climates, and, second, to order the information such that it can be analyzed and presented to practical construction engineers and managers. The first and second tasks may be done simultaneously by developing an orderly hierarchal taxonomy, which we have developed and presented here.

ACKNOWLEDGMENTS

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Snow and Ice Control Environmental Best Management Practices

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Abstract

There is a need for the identification of environmental best management practices (BMPs) to aid in responsible and cost-effective operation of winter maintenance programs. State Departments of Transportation (DOTs) are continually challenged to provide a high level of service on winter roadways and improve safety and mobility in a cost-effective manner, while at the same time minimizing adverse effects to the environment, vehicles, and transportation infrastructure. Understanding and minimizing the negative impacts of deicers is critical to effective and responsible winter maintenance operations. Increasing contamination from the continued use of snow and ice control products has become a significant environmental concern that has detrimental effects on air, water, soil, vegetation, humans, and wildlife. Snow and ice control BMPs have been developed in all aspects of winter maintenance operations. This paper will focus on snow and ice control BMPs with the greatest potential to decrease the environmental impacts of winter maintenance operations including proper material storage and handling, applying the right amount of material in the right place at the right time, and facility management and post storm clean-up. By utilizing the identified best practices, an organization can realize cost and material saving, reduced person and vehicle hours and emissions, and realize a reduction in the impacts of snow and ice control operations on the environment.

INTRODUCTION

There are many ways to describe environmental best practices, including sustainability, green, context sensitive, environmental stewardship, and the triple bottom line. These terms describe an idea or practice, and actions to be taken that are important and relevant to snow and ice control operations. Sustainability is defined as meeting the needs of the present without compromising the ability of future generations to meet their own needs. Therefore, sustainable snow and ice control practices should minimize their environmental footprint. For example, material application methods such as anti-icing or pre-wetting solids allow for the minimum amount of product to be applied, in effect putting less product into the environment and therefore creating less of an environmental impact. Similarly, improved material storage and handling, and enhanced road weather information all contribute to environmental best management practices that minimize the potential risk associated with snow and ice control operations. Minimizing the impacts of snow and ice control operations generally reduces the need for clean-up and mitigation down the road.

The initial movement toward sustainability for transportation agencies was driven by compliance (McVoy et al., 2012). Work by Joen and Amekudzi (2005) found that almost all DOTs have the concept of sustainability in their mission statement, capturing the attributes of system effectiveness and efficiency, and system impacts on the economy, environment, and social quality of life. McVoy et al. (2012) found that most snow and ice control professionals feel that they have a good understanding of how their work impacts the environment and have resources available to them on this topic. This past year at the 2013 Winter Maintenance Peer Exchange, members of the audience were challenged to think of traditional winter maintenance best practices in a new light (Pape, 2013). Traditionally snow and ice control best practices have been seen as practices that save material, costs, and person hours; and provide increased efficiency through the use of equipment, technology, and management. Yet, implementation and acceptance of many of these practices has moved at a glacial rate because of resistance to change, risk aversion, lack of training, and lack of appropriate resources. However, many of the technologies, techniques, and ideas used as best practices have been studied, and information is available on their effectiveness. For snow and ice control professionals to move forward with adopting best practices it is critical to communicate successes and failures, encourage Project Champions, conduct cost-benefit studies, take the risk of implementing unconventional practices, set small attainable goals, and continually working on improvements – before, during, and after implementation.

Three areas where best practices have been identified and shown quantifiable reduction in the environmental impacts of snow and ice control practices including proper storage of solid and liquid snow and ice control products; application of the appropriate material, in the right place, at the right time; and appropriate facility management and post storm clean-up. Information presented on the above mentioned best practices was gathered using an extensive literature review, and survey and interviews of winter maintenance practitioners.

PROPER MATERIAL STORAGE

Proper storage of snow and ice control materials is a key component of managing an efficient and environmentally sensitive winter maintenance program. Effective storage practices save material from being lost to erosion, keep the material workable, and prevent it from leaving the site as runoff and impacting the surrounding soil, plants, ground and surface water (Salt Institute, 2013; TAC, 2013). Wind can blow material off piles and off site, and rain can cause a loss of 0.25% of a salt pile per annual inch of precipitation (US EPA, 1974). At the same time, snow and ice control product storage facilities have the greatest potential to impact the environment, because they are a single source that can release high concentration runoff into the environment (OWRC, 2012). The potential environmental impacts of these products decrease as they are applied to roadways, because the material is widely distributed across the driving lanes and over many miles, and the runoff can be further diluted by snow and ice (OWRC, 2012).

The following best practices for snow and ice control material storage were published by the US EPA's *Manual of Deicing Chemicals: Storage and Handling* (1974):

- Covered storage of salt and other deicing chemicals is strongly recommended using permanent structures.
- General precautions and good housekeeping practices should be used.
- Personnel who administer and supervise highway maintenance should be knowledgeable of their environmental responsibilities.

State DOTs have had access to material storage best practices for over 40 years. However, recent survey results on salt storage facilities found that 3 to 75% (26% median value) of inventoried salt storage facilities used by DOTs and counties are still uncovered, and product may also be stored on unsealed pads, or have insufficient covering (Walsh et al., 2014). Clearly, there is still a need to present and disseminate best practices on material storage. If all snow and ice control materials were stored using known best practices, contamination of surface and ground water from storage sites could be significantly reduced if not eliminated.

Solid storage. There are many options for material storage, which range in their ability to prevent material loss, as well as in price. Options include bins, pads, traditional domes, rectangular sheds or barns, high arch structures, and silos; and can be constructed from wood, steel, aluminum, fiberglass, concrete, or fabric (TAC, 2013). The Salt Institute (2013) developed a reference guide for pile size and storage capacity of buildings, loading and storage styles, and techniques.

Salt storage structures should have at a minimum an impermeable pad and a waterproof covering (OWRC, 2012; Wisconsin DOT 2013). Additional storage facility features to consider include impervious walls that are least three feet high, or one foot higher than the stored product (salt) contact zone, and the walls should be free of gaps or cracks and sealed. Some practical considerations include noting the prevailing winter wind direction and positioning the building and doors with regard to sheltering loading operations, minimizing snow drifting around doorways, and keeping precipitation out of the storage areas (TAC, 2013). Other best practices include promoting indoor operation of all snow and ice control material handling activities where possible (OWRC, 2012; TAC, 2013), and routine storage facility maintenance including checking for roof leaks, tears, or damage, and repairing them in a timely manner (OWRC, 2012).

In a survey by Fay et al. (in press) 96% of responding transportation agencies stated that they store salt or salt-sand mixes on a paved surface inside a covered storage structure, but one respondent commented that they do still have some outdoor salt-sand piles that they are working to cover.

Liquid storage. Liquid storage tanks should be protected from vehicle impacts, and should be installed with secondary containment around the storage tanks and liquid transfer points (TAC, 2013). Secondary containment for liquid storage tanks may be double walled tanks and or containment dikes (Michigan DEQ, 2007; TAC, 2013). Note that double walled tanks, will not protect against leaks from broken fittings or hoses. Recommended secondary containment capacities are in the range of 100 – 125% of the capacity of the largest tank, or 10% of total tank capacity (Michigan DEQ, 2007; TAC, 2013). All valves, hoses, and pumps should be located within the secondary containment. Spills of liquid snow and ice control products can occur during production, delivery, transfer to the spreaders, and in storage if hoses and fittings break. Designers should consult with local environmental regulatory authorities regarding siting and containment requirements for liquid storage facilities (TAC, 2013).

Liquid products should be stored in well-maintained and labeled storage tanks (OWRC, 2012). Scheduled maintenance should be performed for all tanks fittings, valves, and pumps, and any leaks should be addressed in a timely manner.

A survey by Fay et al. (in press) found that while all responding agencies stored liquid products on site only 25% had secondary containment, while 50% responded that some but not all liquid storage had secondary containment, and the remaining has no secondary containment. This could also be stated as 75% of respondents had some form of secondary containment at some but not all storage sites. Feedback from respondents indicated that secondary containment was generally only used for liquid storage tanks.

APPLICATION OF THE APPROPRIATE MATERIAL, IN THE RIGHT PLACE, AT THE RIGHT TIME

Selection of appropriate snow and ice control materials. Snow and ice control chemicals are select based on the type of initial precipitation type and time of onset, expected intensity, total precipitation, air and pavement temperature, expected temperature changes, and pavement condition at the time of onset (Fay et al., in press). Additionally other factors such as product performance, cost, and impacts to the environment and infrastructure are considered. Material application best practices consist of selecting the most effective snow and ice control material for a given road weather scenario combined with the most efficient application method to reduce loss of material, clean-up costs, application rates, and frequency of applications. The use of best practices will reduce the environmental impacts of winter maintenance operations by ensuring the least amount of product is used to achieve the desired level of service.

Chloride salts, also known as road salts including sodium chloride (NaCl), magnesium chloride, ($MgCl_2$), and calcium chloride ($CaCl_2$), are the most widely used snow and ice control chemicals (Fay et al., 2008; Shi et al., 2009). Other snow and ice control products are used to a lesser extent including acetates, agriculturally derived products (often added to chloride products), and glycol or glycerin products. These products may be selected for use in a salt sensitive area because they often do not contain chlorides or contain fewer chlorides, but these products may exert other impacts on environment. These chemicals can be used in solid form for deicing purposes, which is the reactive application of chemicals to roadways to break the bond between the ice and pavement. Common application rates for solid chemicals range from 100 – 1000 pounds per lane mile (lbs/l-m) depending on local road and weather conditions (Fay et al., in press) (Table 1). The use of solid chemicals for deicing improves road conditions, allows for higher traffic speeds, and reduces fuel consumption compared to only plowing. Furthermore, the use of solid snow and ice chemicals reduces the need for abrasives, which improves air quality (Cuelho et al., 2010). The effectiveness of deicing greatly depends on the chemicals and the equipment used by the winter maintenance agency (Chappelow et al., 1992). Solid chemicals are the most effective option on thicker snow accumulations.

Pre-wetting is the method of adding liquid chemicals to solid salts or abrasives prior to application on roadways, which has been shown to improve distribution on roadways, increase performance, and minimize bounce and scatter, thus reducing the amount of material required (Hossain et al., 1997; Michigan DOT, 2012). Commonly used application rates for pre-wetting range from 8 – 20 (gallons per lane mile (gals/l-m) (Fay et al., in press) (Table 1). Dry salt and abrasives on roadways can be lost to the effects of wind, traffic, and bounce prior to actively melting snow and ice. By contrast, pre-wet solids adhere to the road surface, increasing longevity on the road and reducing the number of applications needed. A case study observed that 96% of the pre-wetted material was retained on the road surface, whereas only 70% of the dry material was retained on the road surface (Michigan DOT, 2012). In another case study, a 15% reduction in product usage was reported when using brine to pre-wet salt, because the pre-wet salt showed high ice melting performance and better adherence to the roadway (Fay et al., 2013). Greenfield (2013) found that for pre-wetting rates ranging from 5 - 20 gal/l-m, lower vehicle speeds during application exhibited a tighter distribution pattern and loss of material significantly decreased.

Table 1. A summary of commonly used snow and ice control products and application rates (Fay et al., in press).

Product Type		Application Rate
Sand		100 - 1000 lbs/l-m
Salt:sand (10 - 50% salt:sand)		400/1000 lbs/l-m
Chlorides	NaCl solid	100 - 800 lbs/l-m
	NaCl liquid	10 - 40 gal/l-m
	NaCl pre-wet	8 - 20 gal/l-m
	$MgCl_2$ & $CaCl_2$ solid	100 - 500 lbs/l-m
	$MgCl_2$ & $CaCl_2$ liquid	10 - 40 gal/l-m
	$MgCl_2$ & $CaCl_2$ pre-wet	8 - 20 gal/l-m

Anti-icing is the proactive application of chemicals to a roadway surface prior to a snow event to prevent the formation of bonded snow and ice to a roadway aiding in snow removal (Fay et al., 2013). Commonly used application rates for anti-icing range from 10 – 40 gal/l-m (Fay et al., in press) (Table 1). The use of anti-icing has been reported to have produced approximately 50% materials savings as compared to the use of solid rock salt (Fay et al., 2013). In addition, Boselly (2001) determined that five times more energy is needed to remove snow and ice from a roadway surface once the bond between snow and ice and the pavement has formed. It has been reported that savings from decreased material usage, labor costs, reduced environmental impacts, and increased levels of service result in overall savings of 10 to 20%, with a 50% reduction in the cost per lane mile for winter maintenance operations (Nixon, 2002). The Colorado DOT reported a reduction of 55% in sand use and a decrease in the cost of winter operations from $5,200 per lane mile to $2,500 per lane mile when anti-icing techniques were used (Cuelho et al., 2010). Therefore, effective anti-icing strategies can reduce the environmental impacts of snow and ice control chemicals by reducing application rates, materials usage, and costs while maintaining a high level of service.

Magnesium chloride and calcium chloride work at colder, or lower, temperatures than sodium chloride (Ketcham et al., 1996; Shi et al., 2009; Warrington, 1998). Therefore, depending on road and weather conditions, material selection is critical to determine the best performing product to minimize material usage and the potential environmental impacts. Additionally, pretreatment or anti-icing is not recommended if blowing snow or rain will occur. Liquid chemicals on the road surface will cause blowing snow to adhere to the roadway surface and rain will dilute or wash away the liquid chemicals (Nixon, 2002). This validates the importance of accurate and timely road weather forecasts for appropriate material selection.

Utilizing weather information for timely application of snow and ice control products. RWIS, Road Weather Information System, is nationwide network of strategically placed meteorological stations that report information on roadway conditions including air and pavement temperature, relative humidity, precipitation type and amount, and additional sensors can be added. RWIS has been well documented through studies such as NCHRP Synthesis 344: Winter Highway Operations and the FHWA Test and Evaluation Project 28: Anti-icing Technology, Field Evaluation Report (Conger, 2005; Ketchum et al., 1998).

Boselly et al. (1993) and Boselly and Ernst (1993) found that the use of RWIS technologies can improve the efficiency and effectiveness, as well as reduce the costs of highway winter maintenance practices. Identified benefits from the use of RWIS include the increased ability to obtain accurate meteorological data and the potential for data dissemination and exchange with other agencies; reduced material and equipment use, as well as reduced staff overtime, less misdirected staff time, and improved roadway level of service (LOS); reduced motorist travel time by reducing traffic volumes, and reductions in traffic crashes, and operating costs (Ballard et al., 2002; Michigan DOT, 2010; Strong and Fay, 2007; Ye et al., 2009).

Numerous studies have reached the consensus that appropriate implementation of RWIS is cost-beneficial. McKeever et al. (1998) developed a life cycle cost-benefit model for RWIS and direct savings (benefits) included reduced winter maintenance costs (patrol, labor, equipment, and materials), while indirect savings included reduced liability risk, accidents, pollution, and travel costs. It was found through the model that a savings of approximately $923,000 could be accrued over a 50 year period. Alberta Infrastructure and Transportation (2006) studied the costs and benefits of deploying RWIS in Alberta (2005-2006) and anticipated that RWIS would deliver $5.38 in benefits for every dollar spent. The estimated cost savings are derived from the efficient use of winter maintenance equipment and de-icing chemicals, reduced rate of vehicle collisions, and efficient use of departmental staff. A study conducted in Idaho found a 40% reduction in material use due to the introduction of RWIS (ITD, 2009). Based on these findings, Veneziano et al. (2014) conducted a costs-benefit analysis of newly deployed RWIS for the state of Iowa, assuming a conservative 15% in material savings. The results estimated a benefit-cost ratio of 3.8 for Iowa DOT alone and a total benefit-cost ratio of 45.4 (Veneziano et al., 2014). A survey conducted by Fay et al. (in press) found that 90% of responding agencies have increased or improved their weather forecast, and RWIS, or weather station network to aid in winter maintenance operations.

A study is currently underway in Italy to examine the environmental benefits of implementing advanced RWIS. The research aims to quantify the impact of deicers on aquatic systems and air quality, and empirically evaluate the environmental gain due to the advanced RWIS. The CLEAN-ROADS project will test a state-of-the-art RWIS fully integrated with a Maintenance Decision Support System (MDSS) along with an environmental monitoring system (Pretto et al., 2014).

Additional tools to aid in the timely application of snow and ice control products include the use of Maintenance Decision Support Systems (MDSS), Fixed Automated Spray Technology (FAST), public and private weather forecasts, and the use of stationary and mobile sensors.

Calibration of equipment to ensure appropriate application of snow and ice control materials. Calibration ensures equipment is operating optimally, so the right amount of material can be applied and accurately accounted for by storm and season. Proper and frequent equipment calibration is a best management practice in itself, and can lead to cost and material savings, as well as reduced product in the environment. Calibration should occur (Blackburn et al., 2009; Fay et al., 2013; TAC, 2013):

- at the time equipment has been acquired or installed,
- prior to the start of each winter season and points throughout the season,
- if material calculations show a discrepancy,
- after major maintenance is performed and after truck hydraulic fluid and filters are replaced,
- after the speed (truck or belt/auger) sensors are replaced,
- after new snow and ice control material is delivered to the maintenance garage location.

Calibration should be completed for each application method for each product type - liquid, solid, and pre-wet products. The equipment that controls the spread pattern should also be calibrated to match the recommended application rates and ensure proper placement only in the travel lane, avoiding scatter or bounce that can lead to material leaving the roadway and impacting the roadside environment (Fay et al., 2013). In conjunction with calibration, spreader and sprayer equipment should be set up so that they are mechanically restricted from applying more than a maximum amount of material approved for a given set of routes (Fay et al., 2013). This will further ensure that excessive amounts of material are not used during anti-icing and deicing operations.

The following benefits from conducting annual salt spreader calibration have been identified: the financial benefit of reduced salt usage, and increased accuracy in budgeting and ordering, identification of units with high salt usage that require either repair or replacement, the calibration information can be used as a performance measure for truck hydraulic systems and electronic controls, accurate information on the amount of chlorides applied within an area or even on a specific snow route, the calibrated equipment ensures a quality baseline for determining cost of service and level of service (Habermann, 2012). A survey conducted by Kimley-Horn (2010) found that agencies which calibrated their spreaders realized an 8 to 14% reduction in salt and grit (sand) use. This survey emphasized that several crews reported less runs to get the same results due to calibration. A survey conducted by Fay et al. (in press) found that 90% of responding agencies calibrate spreader equipment annually, but when asked specifically if liquid application equipment is calibrated annually 76% responded that it was. A follow-up question asked if responding agencies had made any effort to improve calibration, and 58% responded yes followed the following comments:

- contractors are now being required to calibrate equipment,
- calibration training is being offered to operators,
- we are performing more frequent calibrations,
- upgrading spreader controllers has simplified the calibration procedure (Fay et al., in press).

FACILITY MANAGEMENT AND POST STORM CLEAN-UP

The design and operation of maintenance facilities can have a direct influence on potential contamination issues and loss of materials. Maintenance facilities may be specific to winter maintenance operations or may be a central location for various maintenance activities. Maintenance facilities should be designed to provide good drainage away from material storage and not be directed towards water supplies (Salt Institute, 2013).

Vehicle washing. When maintenance equipment is washed, the wash water may contain dirt, salt, oil, and or grease. Therefore, the wash water needs to be captured and directed into a storage tank. If local regulations allow the use of this water in the production of salt brine, the wash water then needs to go through a properly designed oil and grit chamber. Otherwise, the wash water needs to be directed to the sewer system or an effective storm water management pond where solids are allowed to settle and the salt concentration can be reduced by dilution from other receiving surface water (TAC, 2013).

Many state DOTs have implemented systems by which the water used to wash their vehicles is recycled and then used to make salt brine onsite instead of using freshwater. Reusing the salt laden truck wash water will allow for material cost saving in making the brine solution and conserve water use (Alleman et al., 2004). Additionally, the amount of salt released as runoff into the local sewer system or the environment may be decreased from this practice. The Colorado DOT, Virginia DOT, and Indiana DOT have successfully pilot tested reusing vehicle wash water for brine making (Alleman et al., 2004; Craver et al., 2008; Salt Institute, 2010; CDOT 2012; Fay et al., 2013).

A survey by Fay et al. (in press) found that 94% of responding agencies wash snow and ice control vehicles and equipment soon after each snow event when possible. Additional comments from survey respondents imply that this value may be optimistic, when in reality their agency may have a vehicle washing policy in place, but that it is not always followed. Survey respondents were also asked if the vehicle wash water is captured before entering the sewer system or open ground, and 50% replied no, 39% responded yes, and 3% indicated that yes it is captured and recycled for brine making or pre-wetting.

Sweeping operations. Once applied abrasives eventually need to be cleaned up, or they can accumulate and clog drains and pipes, get crushed by vehicle traffic and become airborne decreasing air quality, or impact the near road environment (soil, vegetation, and water) (Fay et al., 2013). In a survey conducted by Veneziano and Fay (in press), 50% of survey respondents indicated that their transportation agencies had an abrasives clean-up program, while the remaining 50% indicated that they did not. The clean-up activities were conducted by agency staff or less common by contracted crews or private firms. The average clean-up cost per mile was $85.66, with reported costs ranging from $62.95 to $120.00 per mile. Disposal costs were indicated as being factored into these figures. Less than 50% of respondents indicated that abrasives were recycled. One agency indicated that the cleaned-up abrasives were reused to maintain shoulders, and another respondent commented that the abrasives were reused by mixing into new stockpiles during the following winter season. Additional benefits or costs information on the use of abrasive provided by survey respondents included:

- broken windshield claims from abrasive use were $5,000 annually.
- their agency was able to save $20,000 to $30,000 annually by removing all abrasives from use on black top roads, using them only on gravel roads. Creating a cost savings of $185 to $277 per lane mile.

Street sweeping can significantly reduce the amount of sand or abrasives released into the surrounding environment due to runoff events. Street sweeping should begin in early spring after runoff but before any substantial rainfall events occur (Staples et al., 2004). Street sweeping equipment includes air sweepers equipped with vacuum systems, which offer efficient removal of fine particulates (Staples et al., 2004). It has been reported that the cost of combined broom and vacuum sweepers can be approximately two times higher than a traditional sweeper alone. However, the operational costs and service life are comparable (Staples et al., 2004).

Materials captured in street sweeping may contain pollutants such as petroleum hydrocarbons and metals, it is recommended to perform tests on the recovered material to determine possible hazards. A pre-approved site (such as a landfill) should be used to dispose of spent abrasives (NYSDOT, 2011), or they can be land-applied, bio-treated, or thermo-treated depending on the contents. However, the reuse or recycling of street sweepings is recommended if local regulations permit. Options include reuse as snow and ice control materials or as construction aggregates (Staples et al., 2004). Results of a study conducted by the Montana Department of Transportation (MDT) indicated that the most cost-effective option for collected traction sand is the reuse of the collected material as abrasives for winter maintenance because low chemical and metals concentrations were found in the recovered sand (Mokwa and Foster, 2013). Other options for recycling of recovered sand include mixing it with gravel or other aggregates to produce materials that can be used in plant mix surfacing, cement treated base, and shoulder gravel or top surfacing. The reuse or recycling of recovered abrasives can reduce costs by reducing the amount of abrasives purchased by winter maintenance agencies and eliminating landfill disposal costs (Mokwa and Foster, 2013).

CONCLUSIONS

A number of studies have established the impacts of winter maintenance materials on the roadside environment, particularly to soils, plant life and aquatics (Mills and Barker, 2002; Hagle, 2002; Akbar, et al., 2006). While proper material storage; application of the right material, at the right time, in the right amount; and use of post storm clean-up have made great strides in reducing or eliminating the environmental impacts of winter maintenance materials, it is impossible at the present time to completely prevent all roadside contamination. The following best practices to reduce the environmental impacts of snow and ice control have been identified.

- For snow and ice control professionals to move forward with adopting best practices it is critical to communicate successes and failures, encourage Project Champions, conduct cost-benefit studies, take the risk of implementing unconventional practices, set small attainable goals, and continually working on improvements – before, during, and after implementation.
- Three areas where best practices have been identified and shown quantifiable reduction in the environmental impacts of snow and ice control practices are 1) proper storage of solid and liquid snow and ice control products; 2) application of the appropriate material, in the right place, at the right time; and 3) appropriate facility management and post storm clean-up.
- If all snow and ice control materials were stored using known best practices, contamination of surface and ground water from storage sites could be significantly reduced if not eliminated.
    - Salt storage structures should have at a minimum an impermeable pad and a waterproof covering. Liquid storage tanks should be protected from vehicle impacts and installed with secondary containment around the storage tanks and liquid transfer points.
- Material application best practices consist of selecting the most effective snow and ice control material for a given road weather scenario combined with the

most efficient application method to reduce loss of material, clean-up costs, application rates, and frequency of applications.
   - Effective anti-icing strategies can reduce the environmental impacts of snow and ice control chemicals by reducing application rates, materials usage, and costs while maintaining a high level of service.
- Appropriate material selection based on road weather conditions is critical to determine the best performing product to minimize material usage and the potential environmental impacts.
- RWIS technologies and weather forecasts can be used to improve the efficiency and effectiveness, as well as reduce the costs of highway winter maintenance practices; by providing accurate meteorological data and the potential for data dissemination and exchange with other agencies; reduced material and equipment use, as well as reduced staff overtime, less misdirected staff time, and improved roadway level of service; reduced motorist travel time by reducing traffic volumes, and reductions in traffic crashes, and operating costs.
- Proper and frequent equipment calibration should be completed, and can lead to cost and material savings, as well as reduced product in the environment.
- The design and operation of maintenance facilities can have a direct influence on potential contamination issues and loss of materials.
- Vehicle washing should be performed and the waste water captured, treated, or recycled.
- Street sweeping can significantly reduce the amount of sand or abrasives released into the surrounding environment. The reuse or recycling of recovered abrasives can reduce costs by reducing the amount of abrasives purchased by winter maintenance agencies and eliminating landfill disposal costs.

Additional resources that provide best practices for snow and ice control operations include US EPA (1993) and Maryland State Highway Administration (2014).

The identified best practices should be considered and efforts made to implement them to address public concerns and complaints regarding contamination of the environment from snow and ice control products, as well as provide cost and material savings, and improve asset management. While it may be impossible to eliminate all contamination associated with winter maintenance practices, particularly treatment using deicers, it is possible to significantly reduce the footprint of winter operation on the adjacent environment

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Experimental Study of a Snow Melting System: State-of-Practice Deicing Technology

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Abstract

This paper presents the experimental study of a snow melting system embedded in concrete sidewalks/slabs at Northern Arizona University (NAU) as a practice of deicing technology. NAU is located in Flagstaff, Arizona. In the winter months with high attitude (7000 feet/2,250 meters), temperatures in Flagstaff are extremely cold and the amount of snowfall (approximately 108 inches/2.74 meters per year) has been significant. As of 2012 NAU has planned to install snow melting systems in sidewalks with the goal to provide visitors, students, faculty, and staff with a stably safe environment for transportation in the snow. Hydronic heating system has been selected to generate heat energy using glycol-water as the heat source. Pipes are embedded 4 ½" (11.5 cm) below the pavement surface with glycol-water being circulated within the pavement. To help with mechanical system design, numerical modeling along with yearly heat output values in Flagstaff were calculated. In January 28, 2013, a snow storm was blasting across the city. The snow melting systems were turned on at midnight to heat the concrete slabs. Based on an observation in the early morning of the following day at 7am, no snow accumulation was found on all heated sidewalks at the three project locations. The observation has validated our heat output computation and numerical analysis, and has proved they were appropriate. The hydronic system embedded in the pavement was working very well to generate heat energy to keep the snow away from the sidewalk surfaces. To date, the snow melting systems installed at NAU have successfully demonstrated their abilities to heat the pavements and keep the snow from the surface.

INTRODUCTION

Pedestrian, bicycle and motor vehicle accidents occur frequently in many cold regions due to the pavements being covered by snow or ice. These snowy or icy conditions on the pavements lead to an increase possibility of injury and damage associated with pedestrians, bicyclists, and drivers who travel on sidewalks, streets, highways, and bridges. As a result, the pavements cannot provide the roadway users with comfortable experience, transportation safety, and transportation efficiency. Therefore, snow melting or de-icing has been a critical practice to keep the roadway surface from snow/ice accumulating. Several deicing technologies such as chemical, mechanical, and thermal systems have been applied in the sidewalks to prevent snow/ice accumulating so as to provide a safer transportation environment. Usually, the mechanical methods involve high labor intensity with slow removal speeds and may interfere with traffic. The mechanical methods include plows, brooms, and sweepers for snow removal from priority areas. It has been observed that snow removal activities can be damaging to the pavement surfaces and imbedded lighting fixtures (Xu et al., 2012). Chemical treatments contain using anti-icing or de-icing chemicals. The most common chemicals include sodium chloride, calcium chloride, magnesium chloride, potassium acetate, and calcium magnesium acetate (Zhang et al., 2009). The chemicals can be applied in the form of either solid particles or liquid spray before or after snowstorm event. The treatment also requires significant amount of labor to apply the chemical agent, and extra time is needed for the chemical agent to be effective. To solve these problems, multiple technologies are researched and utilized. Started from 90`s, in-place anti-icing system are being used to remotely apply chemicals to the area that has high traffic volume. More recently, Nixon (2006) has researched and tested a snow-melting technology of surface overlay with special designed aggregated. The aggregate has a sponge-like manner that will retain the anti-icing chemical liquid for a period of time and gradually releases them onto the surface.

In addition to current mechanical methods and chemical treatments, thermal systems known as snow melting systems have been considered as an effective technology for snow removal and improvement to pedestrian and bicyclist safety. Tuan (2004) from the University of Nebraska-Lincoln researched conductive concrete. By adding steel fiber and shavings into the concrete mixture, electrical resistance heating could be generated to prevent ice formation. More recently, Yang et al. (2012) used carbon fiber tape as a deicing heating panel to generate energy. While the application of this carbon fiber based technology is still under review by the market, in order to approve its appropriateness, wide array of research and practice activities are needed. Other thermal systems utilize infrared heating or microwave and radio frequency power, these systems could be operated by truck-mounted system (Zhang et al., 2009). However, In North Arizona, no highway agencies have adopted a snow melting system as a part of deicing technology due to limited research and practice available for decision making. One of the concern is the effectiveness of snow melting systems as opposed to the traditional snow removal methods (i.e., snow plows, deicer, etc.). This paper serving as a pilot study introduces the design process of a snow melting system and its applications in concrete sidewalks on the Northern Arizona University (NAU) campus. The objective of this paper is to present a state-of-practice snow melting system and explains how this system has been applied at three locations on campus as a part of deicing technologies to improve the transportation safety in the winter months.

DESIGN OF SNOW MELTING SYSTEMS

Pavement heating facilities known as snow melting systems have been widely used worldwide. These snow melting systems can be installed in sidewalks, roadways, and bridge slabs to generate heat energy and warm the surface. As an alternative to the traditional snow and ice removal methods, heated pavement has been researched, experimented, and installed in many countries in the past 50 years. By installing heating facilities into the pavement, heat can be released when low temperatures and moistures are detected, thus preventing or melting the accumulated snow and ice. The most important advantages of the heated pavement system include reducing environmental impacts of chemical de-icers, and labor-based snow removal activities.

The most common practices of the heated pavement are achieved by installing either electrical system or hydronic system in the pavement structure. In early 1960's, electric cables were first installed in an approaching slab to a highway bridge in New Jersey (Henderson 1963). However, this electric based heating source was later destroyed and abandoned due to a highway asphalt overlay project and significant pavement displacement. As opposed to the drawback of electric cables, hydronic system is the most promising alternative for heated pavement based on its higher energy efficiency and strong system controllability (Xu et al. 2007). In the hydronic system, pipes or tubes are embedded in the pavement using glycol solution, hot water, or steam, being circulated in pipes to warm the pavements and melt the snow. The pipe materials for the hydronic system vary including plastic, steel, iron and copper pipes embedded within the pavements. However, considering corrosion issue and environmental impact, plastic pipes have been used in most of the heated pavement practices in U.S. The obvious benefits of using heating facilities in the pavements is to eliminate the need for snow removal, provide greater safety for pedestrians, bicyclists, and vehicles, and reduce the labor of slush removal (Lund, 2013).

HYDRONICALL HEATED SIDEWALKS AT NORTHERN ARIZONA UNIVERSITY

Geographic overview. Northern Arizona University (NAU) is located in Flagstaff, Arizona. The City has been well known for its unique attraction and severe weather conditions. According to the City of Flagstaff, Flagstaff's annual snowfall is approximately 108.2 inches (Webpage, 2013). In the winter months with high attitude (7,000 feet/2250 meters), temperatures in Flagstaff are extremely cold and the daily minimum temperature is often below freezing (Table 1). As can be seen in Table 1, each year snow comes to Flagstaff beginning in the early October and spanning to the following middle May. The significant amount of snowfall and ice build-up on pavements in Flagstaff has made snow removal an important task to ensure the safety of pedestrians, bicyclists, and motor vehicles. Neither city government nor Arizona

Department of Transportation Flagstaff Regional Office has adopted any snow melting system as a deicing alternative. It is due to unavailable research activity on the performance of a snow melting system in street pavements and/or sidewalks. As of 2012, the Department of Facility Services at NAU has embarked on the use of snow melting system in sidewalks to provide visitors, students, faculty, and staff with a stably safe environment for transportation in the winter. The details will be presented in the next section.

Table 1. Temperature data in Flagstaff.

Flagstaff Temperature	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Avg. Temperature (°F)	28.7	31.5	35.3	42.3	50.4	59.8	66.3	64.1	57.3	47.2	36.8	29.6	45.8
Avg. Max Temperature (°F)	42.2	45.3	49.2	57.8	67.4	78.2	81.9	79.3	73.2	63.4	51.1	43.3	61
Avg. Min Temperature (°F)	15.2	17.7	21.3	26.7	33.3	41.4	50.5	48.9	41.2	31	22.4	15.8	30.5
Days with Max Temp of 90 F or Higher	0	0	0	0	0	1	2	< 0.5	< 0.5	0	0	0	3
Days with Min Temp Below Freezing	30	28	30	25	14	3	< 0.5	< 0.5	3	18	28	30	209
Monthly Snowfall (inches)	20.4	18.4	22	9.9	1.7	< 0.05	< 0.05	0	0.1	2	9.9	15.9	100

Source: NOAA (2013)

Design of hydronically heated sidewalks at Northern Arizona University. Northern Arizona University (NAU) has installed hydronically heated snow melting system in sidewalks on campus since 2012. Prior to the installation, an initial screening and evaluation was performed and then three locations were chosen, the Social and Behavioral Science (SBS) building, Engineering Building, and the Suites Student Apartment Bridge (Figure 1). These buildings were selected based on the geometric locations where most pedestrians and bicyclists travel to and from their classrooms, and injuries and accidents were repeatedly occurred in the past. As mentioned previously, each year snow comes across the campus from early October to the following middle May bringing a significant amount of snowfall and resulting in an icy and slippery condition on pavement surfaces. The NAU Department of Facility Services staff members have worked diligently to keep sidewalk surfaces away from snow accumulation in the winter months. However, due to an intensive labor need to perform snow removal across the campus (735 acres), it is difficult to completely maintain the transportation safety and efficiency, particularly in the early morning at 6am when some students, faculty, and staff members come to their classes and offices. Thus, the university administration has decided to install hydronic heating systems in the sidewalks to facilitate the improvement of pedestrian and bicyclist safety during the snow seasons.

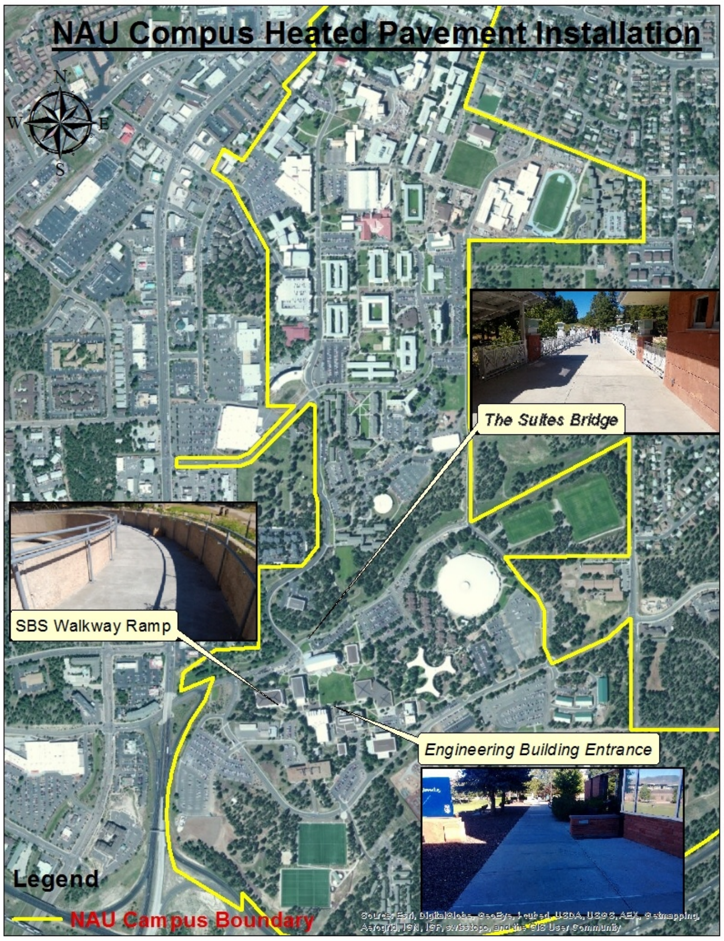

Figure 1. Map of hydronically heated sidewalk installations on campus.

Various reasons surround why the university has decided to use a hydronic snow melting system instead of an electric snow melting system. Among which the most critical reason was due to the minimum performance requirements for the design, construction, inspection and maintenance of the system for use. Energy sources such as direct-use of geothermal hot water, underground thermal energy storage (UTES), boilers and heat exchangers are considered to be used for a hydronic snow melting system. The best type of source to be used depends on specific geological, hydrological, and available land for the proposed site. The energy source of NAU heated pavement at three locations, as shown in Figure 1, is from NAU South Heating and Cooling Plant. This plant is conveniently located at south campus and was designed as the heating and hot water source for the student housings and buildings at NAU south campus. The distances from the plant to the heated pavement installations are all within 300 meters. The university operates the system at 1103 kpa, 120^{o} C supply and 99^{o} C returns, which provide enough heat sources for all buildings and also the heat pavements. With the existing heating facility on-site, the operating cost for the heated pavements could be minimized.

A snow melting project installed in the SBS walkway ramp is used as an example to demonstrate its abilities to provide a stably safe environment for roadway users. A schematic plan of a snow melting system is shown in Figure 2. A heat exchanger is used with the installation of horizontal and vertical ground loops in the pavement. The heat exchanger separates the pavement piping energy-generating system from the liquid source and as a result prevents the pipes and tubes from rust or from external effects. The heat source for the glycol-water snow melting system pump and heat exchanger was used and located at the mechanical room. The heat exchanger transfers heat from the mechanical room to the south ramp of the SBS building where the snow melting system is installed. The mechanical plan of the hydronic heating system include the following primary components are: a 440’-0” (135 m) ground glycol supply and 2” (50.8 mm) diameter return piping system, 7.5 HP pump, heat exchanger, air separator, expansion tank, glycol make-up package, a 1 ¾” tubing snow melting circuiting and standalone air pressure controller serving the snow melting system. Pipes are installed 4 ½” (11.5 cm) below the pavement surface. The area of the heated pavement is approximately 980 square feet (91 sq m), which is also the area of the ramp. The entire cost of the hydronic system installation project in the SBS Building was approximately $160,000 including planning, design, and construction. Thus, the unit cost of the heated pavement is approximately $163 per square feet or $1760 per square meter. Note that two other projects, even though are not mentioned in the paper, are otherwise designed using the same approach.

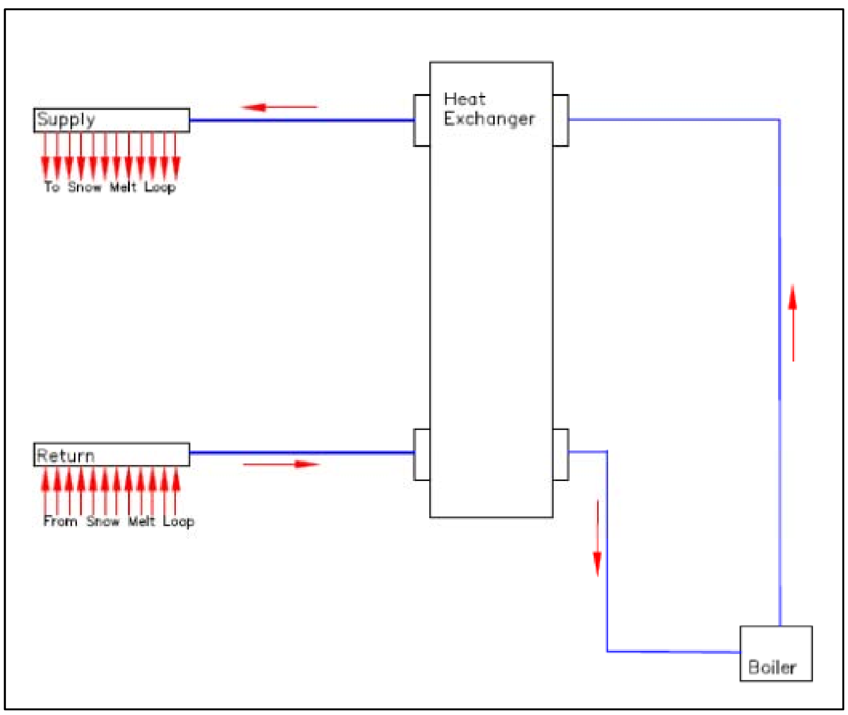

Figure 2. Schematic layout of a snow melting system.

Calculation of heat output and numerical model simulation for design of heating facilities. The capability of heating facilities is designed based on the calculation of heat output for a hydronic system in Flagstaff. In the 1940's, hydronic system has been applied in many countries as a snow melting system. Since then, a number of models of snow melting system on hydronically heated pavements have been presented (Chapman and Katunich, 1957), (ASHRAE, 1995), (Liu et al. 2007), (Nagai and Miyamoto, 2008), (Wang et al. 2010), (Yoshitake et al. 2011) (Xu et al. 2012). According to the ASHRAE Handbook (1995), the heating requirement for snow melting depends on four atmospheric factors: (i) rate of snow fall, (ii) air temperature, (iii) relative humidity, and (V) wind velocity. Given that melting the snow and evaporating the resulting water are the two primary goals for heated pavement to achieve, the air temperature and rate of snowfall will be used to determine the amount of heat required to bring up the pavement temperature to above the freezing point (0°C or 32°F) in assisting with snow melting. The relative humidity and wind velocity will then be used to calculate the rate of evaporation for the melted snow.

Chapman and Katunich (1957) developed a one-dimensional equation to explain the heating requirement of a snow-melting system expressed as:

$$q_o = q_s + q_m + A_r(q_e + q_h) \quad [1]$$

where

q_o= heat output in $Btu/h \cdot ft^2$
q_s= sensible heat transferred to snow ($Btu/h \cdot ft^2$)
q_m= heat of fusion ($Btu/h \cdot ft^2$)
A_r= ratio of snow-free area to total area
q_e= heat of evaporation ($Btu/h \cdot ft^2$)
q_h= heat transfer by convection and radiation ($Btu/h \cdot ft^2$)

Lund (2013) computed the above equations using known/given data and re-arranged the following relations:

$$q_s = 2.6 \cdot s \cdot (32 - t_a) \quad [2]$$
$$q_m = 746 \cdot s$$
$$q_e = h_{fg}(0.0201V + 0.055)(0.188 - p_{av}), \text{ and} \quad [3]$$
$$q_h = 11.4(0.0201V + 0.055)(33 - t_a) \quad [4]$$

where

s = rate of snowfall (inches of water equivalent per hour)
t_a= air temperature (F)
h_{fg}= heat of evaporation at the film temperature (Btu/lb)
V= wind speed (mph), and
p_{av}= vapor pressure of moist air (inches of mercury)

The next step is to determine heat energy needed for a snow melting system to be used in Flagstaff. To do so, parameters in Eq. 2 to Eq. 4 need to be solved. Climatic parameters (V and t_a) in Flagstaff can be obtained from the National Oceanic and Atmospheric Administration dataset (2013). Constant parameters (s, h_{fg}, and p_{av}) were determined using the ASHRAE Handbook (1995) as: s=0.375,

h_{fg}=144, and p_{av}=0.16. With these solved parameters and all given climatic data in Table 1, pavement heat output (q_o) values from October to the following May in Flagstaff were calculated and presented in Figure 3. Based on heat analysis, the snow melting system in sidewalks is estimated to have to supply heat energy at a maximum demand in January (292.82 $Btu/h \cdot ft^2$).

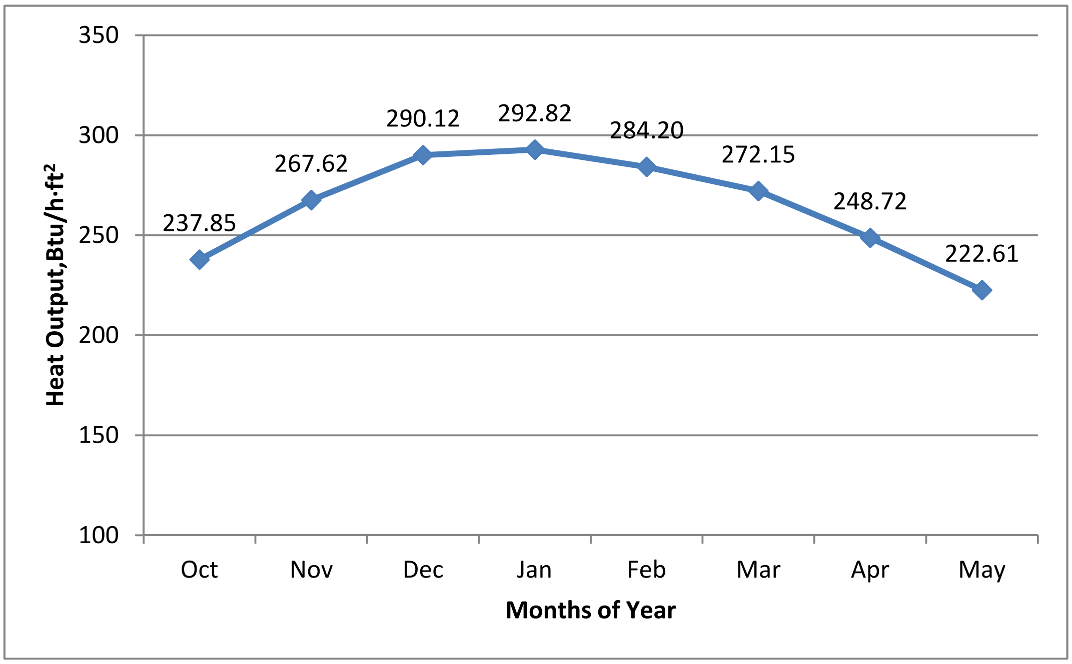

Figure 3. Heat output calculation in Flagstaff.

Given the heat source requirement of the hydronic system, a three-dimensional model was built to evaluate the internal temperature distributions and variations within a concrete sidewalk slab (Figure 4). Based on design criteria, the snow melting system will be turned on when the air temperature reaches at 38°F (3.33°C). The heat source for the glycol-water snow melting system pump has the ability to generate heat energy at a temperature of 140°F (60°C) in the individual heating pipes. To simulate the temperature variations within a sidewalk slab, three dimensional heat transfer model was used to evaluate how the heat source from the heating pipes can bring the heating energy to the sidewalk surface under a cold air temperature of 38°F (3.33°C). In addition, a wind velocity of 10 mph (16.1 km/h) was taken account for the determination of temperature gradients in a concrete slab. Based on the analysis result in Figure 5, it is noticed that the critical temperature was estimated at 53.7°F (12.06°C) at the surface of a concrete sidewalk, provided a surface temperature is sufficient to avoid snow built-up.

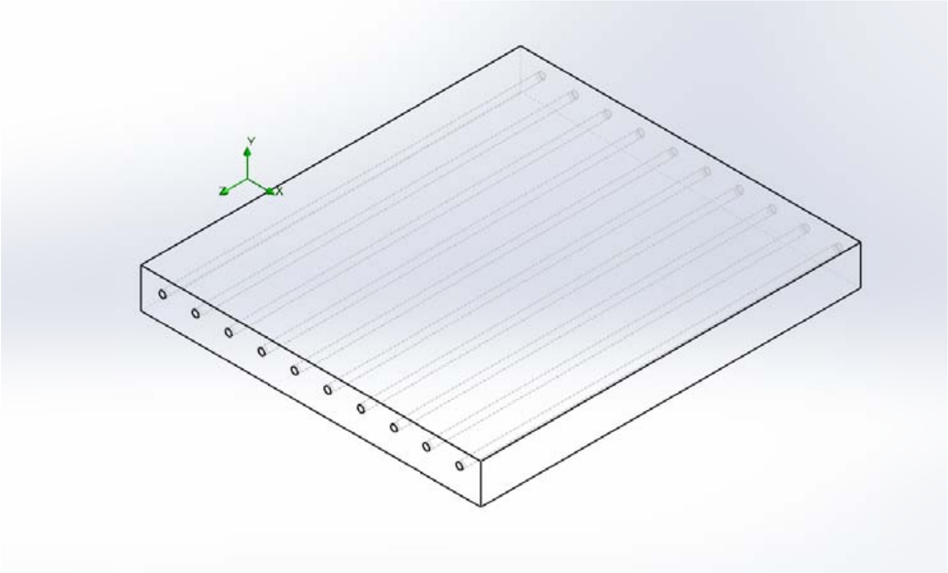

Figure 4. Three-dimensional model of a hydronically heated concrete slab.

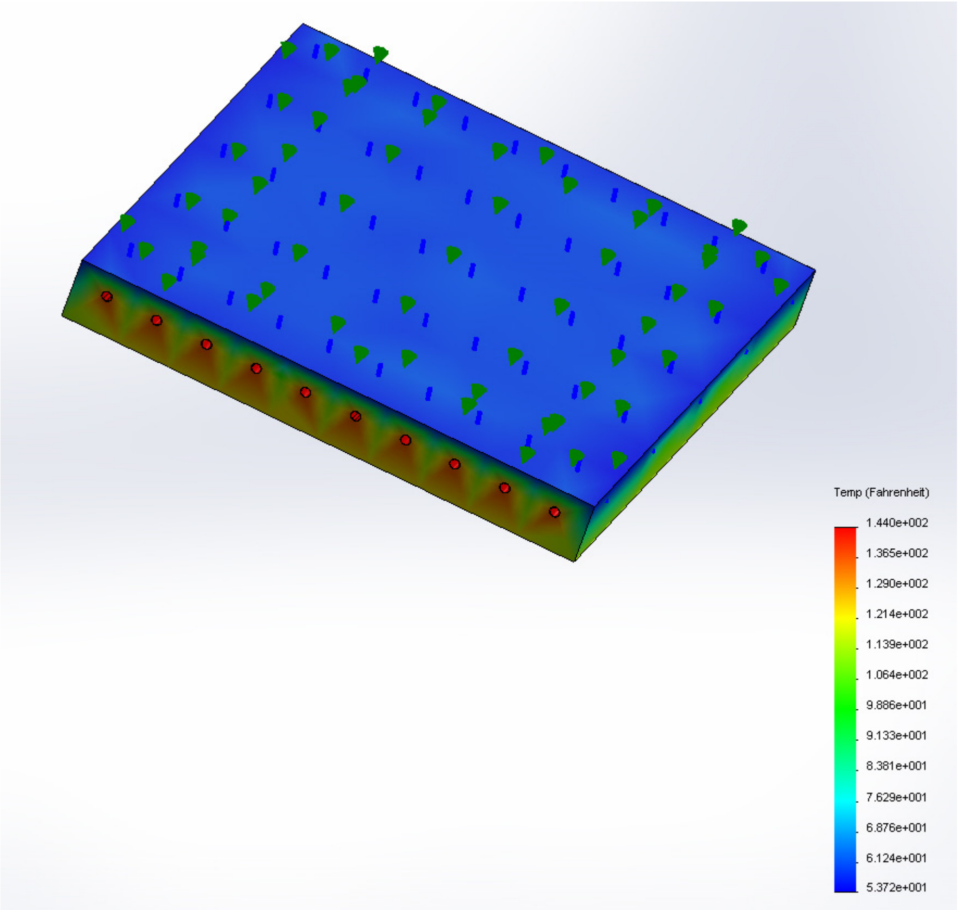

Figure 5. The determination of temperature gradients with addition of wind velocity.

PRACTICE OF SNOW MELTING SYSTEMS ON NAU CAMPUS

From a long-term point of view, the Department of Facility Services at NAU has planned to install hydronically heated snow melting systems in the main sidewalks across the campus. As of today, heated sidewalks have been installed primarily on the south campus including the walkway ramp of the SBS building, sidewalks abound the Engineering Building, and the Suites bridge decks over McConnell drive (Figure 1). The effectiveness of the heating facilities in sidewalks at these locations has been significant in helping with reducing snow removal tasks as well as increasing pedestrian and bicyclist safety in the winter months.

Figure 6 and Figure 7 demonstrate the applicability of hydronically heated sidewalks on campus. These photos were taken on January 28, 2013 in the early morning at 7am. On January 27, 2013, a snow storm was forecasted and expected to bring 2-3 inches (50.8-76.2 mm) snowfall across the city on January 28. After snow storm, the weather data on January 28, 2013 were collected and shown as:

Maximum temperature=34°F (1.11°C)
Minimum temperature = 18°F (-7.78°C)
Average temperature = 26°F (-3.33°C)
Average wind speed= 12.8 mph (20.6 km/h)
Maximum wind speed= 28 mph (45.1 km/h)
Snowfall=2.4 inches (61 mm)

In anticipation of 2-3 inches (50.8-76.2 mm) snow built-up on campus, the Department of Facility Services and the Office of the Suites Student Apartment had planned to turn on the snow melting systems to heat the pavements in the late evening. Snow storm was blasting through the city from the midnight to around 6am with around 1.5 inches (38.1 mm) snow built up measured at 7am. It can be seen in Figure 6 and Figure 7, as of early morning at 7am, there was no snow accumulation observed on the sidewalks from the Engineering Building to the Suites bridge deck. Obviously, the hydronic system within the sidewalks has successfully demonstrated its abilities to heat the concrete sidewalks so that the snow was immediately melted without accumulation. It also confirms that heat out computation is appropriate to help with the mechanical design for a snow melting system. While pedestrians and bicyclists still had to travel onto the sidewalks with caution because of wet surface, the heated pavements have provided roadway users with more comfortable traveling experience in the snowing day.

Figure 6. Heating sidewalks in front of Engineering Building, taken at 7am (01/28/2013).

Figure 7. Heating and non-heating bridge deck comparison at the Suites Student Apartment, taken at 7am (01/28/2013).

For comparison purposes, the sidewalk surface maintained by a snow melting system and labor-based snow removal is explained in Figure 8. The photo was taken on January 28, 2013 at 10 am when the snow stopped. A maintenance crew was able to shave snow built-up out of a sidewalk connecting to the Health Science Building (HSB). Clearly, the snow on the sidewalk to the HSB could not be completely removed due to snow and ice bonding effect. In comparison with the sidewalk on the east wing of the Engineering Building, the pavement surface heated by a snow melting system was clean (even though in a wet condition) and no snow built up was observed. It was anticipated that pedestrians who walked towards the HSB must be very cautious because they might be potentially injured. This comparison in Figure 8 explains the benefit of using a snow melting system in sidewalks in prevention of injury accidents and improvement of transportation safety and efficiency.

Figure 8. Comparison between heating and non-heating sidewalks on the east wing of Engineering Building taken at 10am (01/28/2013).

After the successful performance of the snow melting system at the first winter season, the University has been pleased to adopt this system to keep the snow away from the sidewalk surfaces as well as to mitigate the workload of labor-based snow removal. In the winter of 2013, this snow melting system has continued to provide a stably safe environment for all faculty, staff, and students. Figure 10 shows an example of heating pavements in the same location in front of the Engineering Building on campus taken on December 4, 2014.

In general, snow melting system have been found to be useful in helping the university administration with reducing labor-based snow removal and improving pedestrian and bicyclist safety. While future heated pavement projects are under evaluation and estimation, the performance of hydronically heated sidewalks has been considered satisfactory. Since this type of snow melting system has not been adopted by local governments and highway agencies as part of deicing technologies in northern Arizona, this experimentation serves as a pilot study that is intended to share the design and practice of snow melting system with transportation agencies and institutions in cold regions that might be interested to consider installing a snow melting system in their pavement systems. It should be noted that the snow melting systems have been installed at NAU since 2012, the costs related to labor assignments and associated maintenance activities have still been collected thus not being able to discuss or compare in this paper. Nevertheless, from the safety stand point, the Department of Facility Services at NAU has viewed that the use of snow melting systems in sidewalks as a useful deicing technology and that the system is worth of investment.

Figure 9. Heating Concrete Slabs in a snow say at 11 am (12/04/2013).

CONCLUSIONS

This paper presents the design of heating facilities (snow melting system) in sidewalks at NAU and its practice in helping with reducing labor-based snow removal and improving pedestrian and bicyclist safety. NAU is located in Flagstaff where temperatures with high attitude (7000 feet/2250 meters) are extremely cold in the winter months. To protect students, faculty, and staff from being injured due to snow built up on sidewalks, the Department of Facility Services at NAU has planned to install a snow melting system in sidewalks across the campus. As of today, the effectiveness of using hydronically heated sidewalks at NAU has been found significant and efficient as a useful deicing method. Through the applicability of snow melting systems in sidewalks on the NAU campus, the following conclusions have been made:

1. Snow melting system has demonstrated its abilities to prevent snow accumulations as well as to improve pedestrian and bicyclist safety in a snowing/icy condition.
2. Based on the design, pipes are embedded in the pavement 4 1/2" below the surface. Glycol-water system was used as heat source being circulated in pipes to warm the pavement and melt the snow. The entire project cost for the SBS building was approximately \$160,000 including planning, design, and construction. Considering the area of heated pavement, the unit cost of the heated pavement is approximately \$163 per square feet or \$1760 per square meter
3. When a hydronic system is being designed, a yearly heat output values from October to the following May in Flagstaff were computed to provide the information to help with the mechanical plan. Based on heat analysis, the snow melting system in sidewalks is estimated to supply heat energy at a maximum demand in January (292.82 $Btu/h \cdot ft^2$).
4. A numerical model was used to simulate the temperature gradients from the embedded pipes to the pavement surface. It was determined that the surface temperature of a concrete sidewalk could be warmed at 53.7°F (12.1°C) as the cold air temperature reached at 28°F (-2.22°C)degrees.
5. When a snow storm was forecasted, the snow melting systems were turned on in the last evening to heat the pavement. Based on an observation in the early morning at 7am, no snow accumulation was found on all heating sidewalks, provided the heat output computation and the pavement temperature estimation were appropriate and the hydronic system embedded in the pavement was working very well to generate heat energy to melt the snow.
6. In North America, no highway agencies have adopted a snow melting system as a part of deicing technology due to limited research and practice available for decision making. Thus, the experience of using heating facilities in sidewalks at NAU is intended to be shared with transportation agencies and institutions in the cold regions.

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The Current Status of Roadways Solar Power Technology: A Review

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Abstract

Shining above, the sun has always been an obvious source of energy and recently it has been used on roadways also to produce electricity. The "Raised pavement markers" (RPMs) have been in use in Oregon for many years. A study by Oregon Department of Transportation (ODOT) in 2004 suggested that the solar powered RPMs can be used on the roadway pavements satisfactorily. This study included laboratory testing and field observations. Light Emitting Diodes (LED) is fixed inside the Solar Raised Pavement Markers (SRPM). They can be used along the sidewalk pathways. Photovoltaic (PV) cells work by using semi-conductors to absorb light and create a flow of electrons. The advantages of using the solar powered roadways are not immediately felt right after the construction. The solar photovoltaic panels can be placed on the sound barrier walls and on bridge structures also. This paper deals with the current status of technology related to the solar panels that are used on roadways, bicycle paths, pedestrian sidewalks and along roadside. This paper also discusses the pros and cons and the feasibility of using the solar energy from the roadway pavements, parking lots, sidewalks, bicycle paths and roadside.

INTRODUCTION

The brightest star that is closer to us in the solar system is the sun. For a very long time, humans worshipped the sun. Without it, life cannot thrive on this planet. Sun's powerful ray of light is always the source of energy for millennia. The whole day, even if it is overcast, the radiation from sun warms earth and this energy can be harnessed to produce electricity.

As the energy needs continue to grow, the surrounding environment is polluted badly by burning the fossil fuels and it is high time that an alternative type of renewable energy has to be sought. Solar energy is such an alternative form of renewable energy that can be used for fulfilling the humans' needs. It is the biggest source of renewable energy on the planet. As the availability of fossil fuels and other non-renewable energy sources slowly diminish; consumers are forced to chase after the alternative sources of energy. Over the years, Solar Energy has penetrated our lives through homes, offices, gardens and streets. Also, it protects the environment and helping us to save money on our electricity bills.

Sun's energy is used by humans every day in many different ways. Of all the applications of the sunlight by humans, the notable invention is photovoltaic cells that convert light energy into electricity. The sun creates its energy through a thermonuclear process that creates heat and electromagnetic radiation. Furthermore, the recent innovations in Solid State Lighting technology have made it possible for the mass adoption of high-efficacy energy efficient Light Emitting Diode (LED) luminaires in conventional lighting applications. When equipped with solar power, LED street lighting offers a unique solution to reduction in electricity bills for the cities and affords safety to the citizens.

Recently, the residential buildings have started using the solar energy to save money and to protect our environment. Now the concept of using solar energy for our roads that we travel on is picking up heat. This concept of using solar power for roads is not new. As early as 2003, the Oregon Department of Transportation demonstrated the use of SRPM on the roads and the first report was published the same year.

For the past 10 years, there has been a growing trend of using solar energy on and around the roadway pavements. Solar panels comprising of photovoltaic cells can be embedded on the pavement to create electricity that can be used for the lighting of roads during night as well as for powering the homes by connecting the solar cells to the national grid of electricity. Also, the solar powered LED pavement markers can be embedded on the pavement. Besides using the photovoltaic solar panels on the road, they can be used along the roadside as well as over the roadways as the roof cover. On either case, electricity can be produced and used for roadway lighting as well as for powering homes.

A solar project saves the cost of fuel, delivery costs, and cost of operating the conventional energy system, because sunlight is freely available. For the past 10 years, tax credits and other incentives by local and federal governments have helped reducing the overall cost of solar energy projects. These cost savings cover the high initial cost of solar panels and the accessories with a reasonable rate of return and manageable risk to the investor. Recently, more and more investors serve the solar market by combining utility cost savings, tax credits and other allowances given by local and federal governments to construct cost effective projects.

SOLAR ENERGY ON THE PAVED AREAS

Solar raised pavement markers (srpm). The SRPM have been in use for more than 20 years in U.K and other European countries. In U.K they are called “LED Road Studs” (as shown in Figure 1) and they contain small solar panels and emit LED light to illuminate roadways during night time. Just a few hours of daylight will provide enough battery power to last days - more than enough to ensure the system operates all year round whatever the weather and wherever the location in the world. The solar panel road studs have been installed in over 120 roads in the U.K and in the Noord-Holland Province of the Netherlands (Pearson, 2006).

Figure 1. variable solar stud or srpm (source: http://www.tuvie.com).

On the national level, as of August 2010, the Washington State Department of Transportation (WSDOT) began a 5 year test of the same technology (Solar Powered LED) along a 2 mile section of State Route 530 that has a history of run-off-the-road collisions.[29] The solar-powered road reflectors that WSDOT has installed contain an LED that will automatically light up under dark conditions to provide an estimated 10 times greater visibility for drivers than the traditional retro-reflective markers. Depending on the results of the tests and available funding, WSDOT will consider investing in additional test areas on other roadway surfaces or situations (Bradley, 2014).

Advantages of SRPMs. SRPMs can be used for the visibility of drivers on both directions of travel on the roadways. Also, advanced guidance could be provided by SRPMS to the passing and no passing zones. Nature of the changing terrain and curves ahead can be alerted to the drivers by the introduction of SRPMs. Improved and sustainable night and adverse weather guidance can be provided (fog, rain, ice, skiff of snow etc.) because of their internal illumination rather than retro reflectivity.

Guidance is maintained during winter months regardless of the condition of the painted markings and they are fully compatible with snow removal operations. Night accidents can be reduced considerably.

Disadvantages of SRPMs. On the east end of the SR 530 test site, current recessed RPM's are not visible/apparent in the rain etc. and their subsurface mounting causes sight distances to be erratic and foreshortened. The grooves readily fill with water or debris from sanding etc. In weather events headlights reflect off surface of water, thereby eliminating their conspicuity altogether, or they become hidden from view by debris.

The reflective surface of the stud is well below the road surface and it takes considerable time after a weather event to clear them and again it becomes ineffective when needed most.

When clear of debris or water, the reflective surfaces are short lived and despite their subsurface mounting, they still require frequent replacement. In addition, the groove foreshortens site distances and they are viable only during good weather. Dynamic loading, high-speeds and lateral forces significantly foreshorten raised pavement marker service life on 2 lanes highways.

On the west end of the SR 530 test site, WSDOT striping crews restriped and marked the section and already the raised pavement markers are missing sometimes for significant distances or individually in ways that detract from their guidance except as a general centerline demarcation.

Other uses of SRPM or Solar studs. SRPMs can be used for Pedestrian pathway delineation, marking, cycle path delineation and route 'way-marking'. They can be used for the demarcation of combined pedestrian and cycle paths so that cyclists and pedestrian are separated. They can also be used in airports – taxiways, pushbacks, fire routes and periphery areas (including by-pass and aprons). Car parks and architectural designs are the other areas where SRPMs can be used. (Dornsife, 2010).

Solar Panels Embedded on the Pavement. The world's first Solar Paneled Pavement (SPP) has been installed at George Washington University in Ashburn, Virginia (US). Designed by Building Integrated Photovoltaics (BIPV) specialists the pavement panels are slip-resistant and semi-translucent for optimum aesthetic appeal and functionality. The SPP generates approximately 400 Wp. In total 27 panels (Figure 2) have been used in the pavement design which has been integrated into a popular Solar Walk between two of the university's buildings, Innovation Hall and Exploration Hall.

Figure 2. Solar panels used on walkway (source: www.gwtoday.gwu.edu).

The Solar Walk was completed in 2012 with design features suggested by current students of the George Washington University. The idea behind SPP is simple: sunlight falling on the road surface is absorbed by solar cells and converted into electricity – the road surface acts as a large solar panel. The electricity generated in this way will find practical applications in street lighting and putting the electric power on the national electricity grid.

The concept of solar panels embedded pavement for producing electricity is an out of the box idea to implement. The "Embedded solar panel pavements" consist of individual panels with three layers: a top layer of high-strength, textured glass that provides traction for vehicles, an array of solar cells beneath that for gathering energy, and a base plate that distributes the collected power (Nancy Balp, 2013).

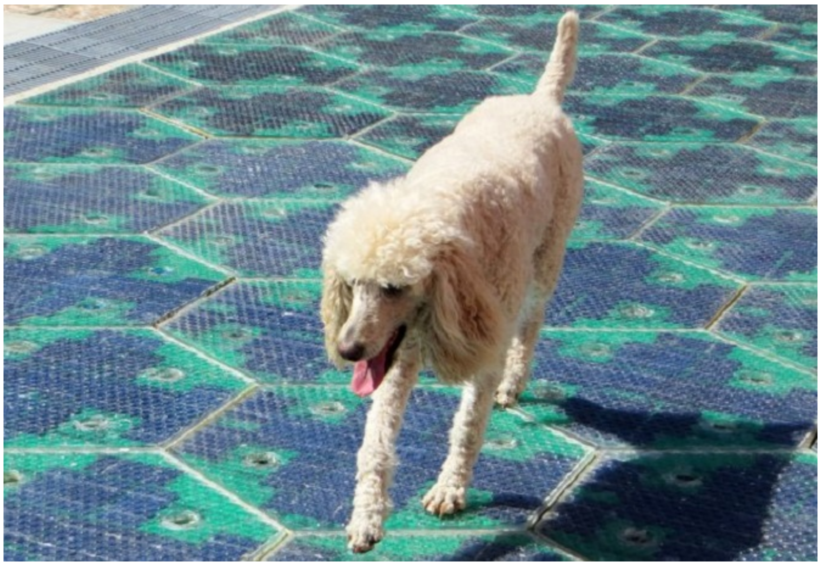

Figure 3. Solar road panels prototype on parking lot (source: http://www.inquisitr.com).

An Idaho-based Solar Roadways, a private company obtained the Federal Highway Administration (FHWA) contract for a project on Solar Road Panel prototype in 2009 (Figure 3). It would integrate features such as LED lights and heating elements into structurally engineered road panels. The glass panels that they used were tested in various University testing laboratories and they all exceeded the properties such as compressive strength, impact strength, friction test, etc. could be similar to the guidelines set forth in the specifications for pavements. It is implied that the glass panels can be manufactured to bear the dynamic loads of trucks, cars, buses and different types of heavy vehicles. Figure 4 depicts a driveway made of solar panels that could be effectively used during day time and that energy can also be connected to the national electric grid (Brusaw, 2014).

Figure 4. Solar panels for driveways (source: www.solarroadways.com).

In Netherlands, a new solar powered bike path (Figure 5) has been opened in the November of 2014. Bike users feel that there is no difference between the regular bike path and the solar powered one. The power generated by this bike path is connected to the national power grid and can be used by the users that are connected by the national grid. The project in the town of Krommenie in Netherlands is being called the world's first public road that includes embedded solar cells.

Figure 5. Solar panels that form the bike path (source: www.bbc.com/capital).

The crystalline silicon solar cells are encased in two layers of tempered safety glass, mounted in a concrete housing. The equipment is part of pre-built concrete slabs that the company says have been refined in years of testing. The company says it's been a challenge to produce energy-producing slabs that are both durable and ride able by thousands of cyclists a day. "It has to be translucent for sunlight and repel dirt as much as possible," the company says. "At the same time, the top layer must be skid resistant and strong enough in order to realize a safe road surface" (Tobal, 2014).

Advantages of Using the Pavements with Embedded Solar Panels. The pavement solar panels can produce electricity by connecting them to the national power grid and hence be used for homes. Carbon footprint on this planet is reduced. They minimize the need for space for electricity generation and distribution. More area is available for use, because the total length of U.S roadways is around 4 million miles.

Disadvantages of Using the Pavements with Embedded Solar Panels. It is more expensive to build the pavements with solar panels rather than without them. Durability of these glass roads are yet to be seen, because it will have to deal with heavy dynamic loads such as Semi-trailer trucks, Construction equipment, buses, heavy machinery and cars. In addition, the cost of repairing these solar panels is likely more expensive than it would be for fixing ordinary, asphalt roads.

The company says it could utilize a type of self-cleaning glass to keep the surface clear of dirt and grime, but this process is not yet proven. Because the solar panels lie flat instead of being angled to take optimal advantage of the sun, the path's panels will produce about 30 percent less power than similar panels might produce on a rooftop.

SOLAR ENERGY ALONG THE PAVED ROADWAYS, BIKE PATHS AND PEDESTRIAN WALKWAYS

Along the roadways, within the Right of Way, the available area can be used for installing solar panels to produce electricity by connecting them to the National grid. It is good that the unused land found great use to produce renewable energy in an efficient way.

Some of the European countries such as Germany, Netherlands and Switzerland have utilized unused spaces on the side of roads for 20 years. But, in the United States although not large in capacity, an interesting solar roadside project has been undertaken in the state of Oregon. In December 2008, the first solar highway project in U.S started in Oregon State that began to produce electricity and it was fed into the national electricity grid. A solar array mounted on the ground with 594 solar panels that produces 104 kilowatt (DC) power is located at the interchange of Interstate 5 and Interstate 205 south of Portland, Oregon. Solar energy created by the solar array feeds into the national electricity grid during the day, in result running the meter backwards for energy needed at night to illuminate the interchange during night through a Solar Power Purchase Agreement with Portland General Electric (PGE) (Oregon Department of Transportation) (Ponder, 2012).

In Europe, the noise barriers are used for installing solar panels to produce electricity. The land strip along a road infrastructure can be used for noise abatement as well as for the placement of PV modules to create electricity. PV on noise barriers does not require special land resources and the costs for mounting structures can be shared between the PV system and the noise barrier. Various PVNB technologies have already been implemented and tested. Particularly the use of bifacial modules may enhance the potential of PVNB. The first photovoltaic noise barrier (PVNB) came into use in Europe in 1989 and is located in Switzerland in the municipality of Domat/Ems. Only a few years later other countries followed that example and build the first PVNB in their country.

Figure 6. Zigzag patterns of solar panels on the sound barrier walls (source: www.photovoltaik.eu).

The glass surface of a PV module can only be applied for sound reflection. In situations where absorption of sound is required a cassette or a zigzag model has to be applied, enabling a combination of sound reflection and sound absorption. Because PV current cells operate most efficiently at lower temperatures great care should be taken in the design of sound absorption cassettes to allow sufficient cooling of the PV modules. In layered or zigzag arrangements (Figure 6), the positioning of modules has to be such that (partial) shading is avoided at all times. Contamination from vehicles on the motorway can have a negative impact on performance if the modules are mounted too low and near the surface of the road.

Successful use of PVNB requires a close cooperation between sound and PV experts, as well as of the road authorities and the financiers of the PV system. To achieve cost reductions for the PV part of the project integrated systems are to be preferred to retrofit systems. With fully integrated PVNB systems, a clear understanding of the cost substitution should be aimed for. In an ideal situation, the noise barrier and the PV plant should be planned and realized at the same time as a single project (Nordmann, 2004).

The first solar project built along a major highway right-of-way on the East Coast of the United States has begun producing energy to power the town of Carver's water-treatment plant in the State of Massachusetts.

Figure 7. Roadside solar panel arrays (source: www.boston.com/news).

Built along Route 44 (as shown in Figure 7) on an easement awarded by the state to the town, the 99-kilowatt project makes use of the highway's east-west orientation to erect an array of south-facing solar panels in a nearly ideal location in terms of power-generating efficiency. The 600-foot-long series of linked panels went on line earlier this month, after NStar examined and approved its connection to the power grid. When the solar array produces more power than needed by the water-treatment plant, the surplus will be sold to the grid. When the plant needs more than the panels are producing, the utility will provide it.

For Carver, a town in Massachusetts, the renewable power supply will save about $3,000 a month in electricity costs. State officials said the Route 44 solar project is a viable good project helping the town while promoting the state's goal of expanding sustainable energy sources (Knox, 2012).

Advantages. Savings on electricity bill can be achieved by using solar panels on the right of way, otherwise not used. Solar panels produce clean energy and it makes use of unused space. More revenue can be made with the extra solar energy that can be used by connecting the panels to the national electric grid.

Disadvantages. They need protection from the theft by criminals. Capital and Maintenance costs need to be monitored. Especially when the panels are deposited with dirt and dust from road traffic use, then they need constant cleaning and additional facilities to prevent theft. Vehicle accidents may cause solar panels gets disturbed.

CONCLUSIONS AND RECOMMENDATIONS

Solar panels can be used on the roadway pavements as well as along the roadside. The pros and cons are to be weighed in before taking decision on the selection of the right type of solar project that can be used on our roadways taking into consideration the different projects that were exhibited in this report.

The advantages and disadvantages of using Roadway pavement solar energy and Roadside solar energy are listed as below:

Advantages. Overall, clean energy, less environmental pollution can be achieved. In the long run of use, cost can be brought down. Safety of the roadway at night can be improved. During the winter period, the icing of pavement and bridges can be avoided. Entire roadway can be lighted with the roadside solar panels creating electricity. If done right, the whole country can be electrified by using solar panels along the roadway.

Disadvantages. Initial and maintenance costs may be very high. When the glass is scratched, it may collect dirt or dust and may become useless because it loses its transparency. Scratch proof glasses are very expensive. During accidents, the repaving may be a very costly affair. To keep the storm water away from roadway is important and it may involve costly procedures to keep the water away. Theft and vandalism may render the roadside solar panels more expensive to afford better protection.

Recommendations. The SRPM has to be improved to provide viability of use under any severe weather conditions. Solar panels can be used on driveways of Single or Multifamily Homes. The solar panels can be installed on the roof of the structure that can cover the entire roadways. They can be embedded on the rails of bridges too.

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Life Cycle Assessment Synthesis for a Volume of Lubricating Oil in Marine Applications

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Abstract

Environmental considerations can be an important part of some decision-making procedures, along with other factors such as cost and risk, when weighing multiple alternative product or process choices. Availability of environmental data is crucial for environmental assessments. In marine vessels, lubricating oil use can vary widely between ships with different filtration systems, engine parameters, and oil change scheduling procedures, all of which can be altered to change lube oil usage. This research addresses a need for environmental data for mineral lubricating oil, in the format of a life cycle assessment (LCA) data synthesis based on a case study for the Washington State Ferries. Data are collected and presented separately for engine oil acquisition, use, and disposal, utilizing cross-platform data sourcing including currently existing literature, ferry oil analysis results with simple combustion modeling, and a software tool database. The purpose of this data assembly is to provide decision-makers with a simple source for the impacts of a typical volume of oil throughout its life cycle. The functional unit used is 3.79 liters (one gallon) of oil, which allows for adjustments for oil use by different systems for equivalent functional comparisons between alternatives, lending widespread applicability.

INTRODUCTION

Lubricating oil is a widely used fluid with a variety of applications in industrial, commercial, and home settings. Engine oils, transmission fluids, and gear oils make up over half of the total lube oil demand in the United States (NP&RA 2005). One of the main functions of engine oil is keeping moving parts within an engine from scraping against one another, reducing inefficient operation due to friction and preventing excessive engine wear (Noria Corporation 2015). However, lubricating oil condition degrades over time, and with use, due to oxidation, viscosity change, wear metal and contaminant build-up, and additive depletion (Livingstone 2013). Therefore, oil must be regularly changed. In marine vessels, lubricating oil use can vary widely between ships with different filtration systems, engine parameters, and oil change scheduling procedures. As a result, each of these can be adjusted by the engine operator in order to change lube oil consumption rate, either through procedural or equipment changes.

Environmental considerations can be an important part of some decision-making processes, along with other factors such as cost and risk, when weighing multiple alternative product or process choices. Lubricating oil has the potential to cause many environmental impacts throughout its life cycle (Epstien and Selber 2002, Endresen 2003, EPA 2011, Riamondi et al. 2012, Geyer et al. 2013, Langfitt and Haselbach 2014). This life cycle includes at least exploration for and extraction of crude oil, refining crude oil into base oil, blending additives into the base oil, use, disposal/recycling, and various transportation processes along the way, as shown in Figure 1 (Langfitt and Haselbach 2014). Each of these stages may have inputs from, and outputs to, the environment such as energy use, resource use, and emissions to air, water, and soils. Given that oil use in marine vessels can be controlled by procedural and equipment selection, it is important to understand the magnitude of environmental impacts associated with lubricating oil use in order to understand what the potential benefits could be from a technological or procedural change that reduces oil use rate.

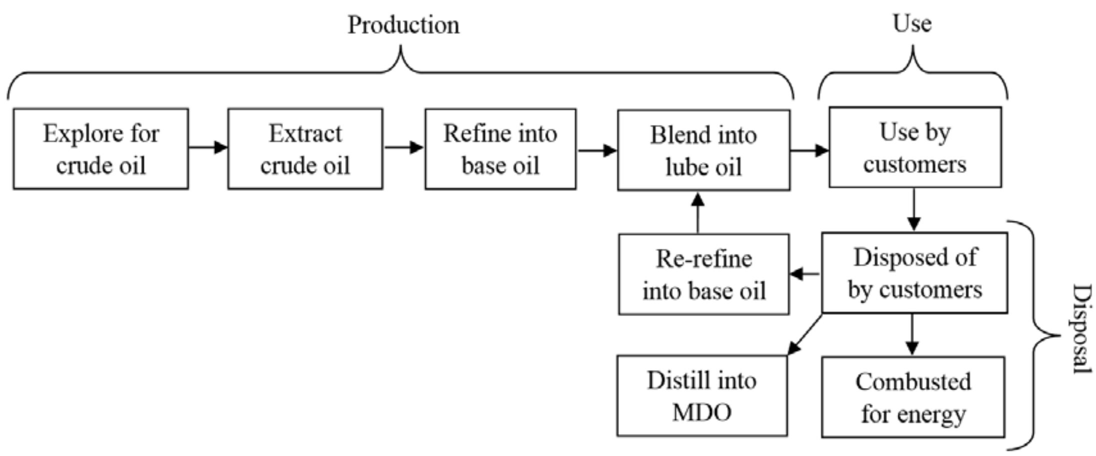

Figure. 1. Life cycle of typical engine lube oil (Langfitt and Haselbach 2014).

Availability of environmental data can significantly affect a company's or agency's ability to utilize environmental assessments during decision-making. This research addresses the need for a straightforward collection of environmental data for mineral lubricating oil, in the format of a life cycle assessment (LCA) data synthesis based on a case study for Washington State Ferries. Life cycle assessment is an internationally standardized procedure for determining the quantities of potential environmental impacts of a product or process throughout its entire life cycle (ISO 2006a). This procedure relies on clear descriptions of goals and methodologies, collection of input and output data based on a functional unit, conversion of that data into a set of potential environmental impacts (with magnitudes), and interpretation of the results. All data presented in this study were either derived from published LCAs or were calculated from inventory data in an attempt to be consistent. Potential environmental impact magnitudes are presented separately for engine oil acquisition, use, and disposal, utilizing cross-platform data sourcing. It appears that prior to the work culminating in this paper (previous results published in Langfitt and Haselbach 2014), no single source existed for quantified environmental impacts of lubricating oil (particularly with a marine vessel focus) over the entire oil life cycle, which drove the need to synthesize this data so that environmental impacts can be considered by fleet operators.

METHODS

Four main steps were followed to carry out this environmental data synthesis including (1) goal and methodological scoping, (2) data collection, (3) data source selection and conversion to functional unit basis, and (4) interpretation and compilation of results. To a large degree, these steps mirror the phases involved in a life cycle assessment (ISO 2006a), although this study should not be considered a life cycle assessment, but rather an environmental data synthesis.

The goal of this study is to provide those in the marine sector with a simple assessment of life cycle environmental oil impacts to utilize during decision-making for decisions which may impact oil use. These scenarios could include at least procedural modifications for oil changes, oil filtration system upgrades, and engine system upgrades. Only one product, mineral lubricating oil, is being examined, so no comparative assertions are made.

Mineral lubricating oil (i.e. from petroleum, not synthetic) is the focus of this study. Specifically, engine oil of this type used to lubricate large diesel engines on vessels was the main goal for data collection. However, most of the data were developed from sources that did not identify the application of the oil, with the exception of combustion modelling that was based on the actual oil properties from a large ferry vessel.

System boundaries are important to defining exactly which processes are included in the assessment. The stages of the oil life cycle included in this assessment are exploration for and extraction of crude oil, refining into base oil, blending, certain aspects of use, and disposal by distillation to marine diesel oil (MDO). In the presentation of results, the acquisition, use, and disposal stages are disaggregated in case changes in usage would occur separately in each stage. Each of these stages derive data from different sources so individual system boundaries, cut-off criteria, and allocation procedures may differ. Transportation processes within a life cycle stage (such as shipping of crude oil to the refinery and transfer of used oil within recycling facilities) are included in the system boundary, whereas transportation processes that are specific to the customer and location (such as shipping new lube oil to the customer or transporting used lube oil to the recycler) are not included since they are not readily generalizable. Technology changes over time, such as in oil production or disposal techniques, are not considered due to their uncertainty and lack of available forecasted data.

A functional unit is “the quantified performance of a product system for use as a reference unit” (ISO 2006a). In this study, the functional unit is 3.79 liters (one gallon) of lubricating oil with a density of 0.88 kg/L. This functional unit was chosen because most engine operators are used to working in oil volume (not mass, like most of the data sources) and because in decision-making processes, differences in oil volume needs may be the most quantifiable measure. However, it should be noted that if systems/procedures are being compared it may be more useful to base this functional unit on some other measure that considers the function of the vessel, such as engine hours, passenger-kilometers, or kg-kilometers.

Life cycle inventory (LCI) data is the quantification of inputs and outputs to the product or process over the life cycle. These data include items such as emissions of NO2 or use of diesel fuel. However, it is difficult to interpret the meaning behind these values because the inventory is generally very large, can include many different units, and the types and severity of resulting environmental impacts are usually not immediately apparent to the reader. Impact categories are sets of environmental concerns that could be caused by these inputs and outputs, and could be midpoint indicators (direct effects, such as potential to destroy stratospheric ozone for ozone depletion) or endpoint indicators (consequences, such as increased skin cancer or crop damage from increased UV radiation for ozone depletion) (Bare et al. 2002). Conversion of LCI data to impact category indicators is accomplished by classifying data into impact types and then characterizing the magnitude of potential impacts caused, in a process defined by the "impact methodology". Many impact methodologies exist (Matthews et al. 2013), however TRACI 2.0 (The Tool for the Reduction and Assessment of Chemical and Other Environmental Impacts version 2.0) is used in this study since it was developed specifically for the United States (Bare 2011), and it uses midpoint indicators which require fewer assumptions and reliance on "less-established facts" (Heijungs and Guinée 2012). In addition, one major data source used this impact methodology (Geyer et al. 2013) without providing inventory data, and using TRACI 2.0 therefore provides more consistency in the interpretation. The newer version, TRACI 2.1, was not used because the update removed toxicity characterization for some metals whose toxicity values in USEtox (one basis of TRACI) are considered "interim" (Geyer et al. 2013), some of which are potentially important in this study as lube oil constituents.

In accordance with ISO 14044 (ISO 2006b), a comprehensive set of impacts categories that reflect many different potential environmental impacts are considered in this study, though there are other potential impacts that are not accounted for. Specifically, the chosen impact categories [units] are:

- Acidification Potential (AP) [H^+ moles-Equiv.]
- Ecotoxicity Potential (ETP) [CTUeco]
- Eutrophication Potential (EP) [kg N-Equiv.]
- Global Warming Potential (GWP) [kg CO_2-Equiv.]
- Human Health Cancer Potential (HHCP) [cases]
- Human Health Non-Cancer Potential (HHNCP) [cases]
- Human Health Criteria Air Potential (HHCAP) [kg PM 10-Equiv.]
- Smog Creation Potential (SCP) [kg O_3-Equiv.]
- Stratospheric Ozone Depletion Potential (ODP) [kg CFC 11-Equiv.]

Data for this study were sourced from currently existing literature, ferry oil analysis results with simple combustion modeling, and the GaBi 6 extension database "XVII: Full US" (GaBi 2014). In all cases preference was given to U.S. data. As previously mentioned, most of these data were not developed specifically from marine applications, so this should be noted as one limitation of the study. Data were collected from multiple sources for stages in which they were available, however, only data from the single source deemed most appropriate for this study by the investigators were used in final reporting, with identification and brief discussion of other sources considered. Some of these data were in the form of LCI data (which were converted to TRACI 2.0 impact category indicators in GaBi 6) while some were already in the form of LCA impact category indicator results.

Normalization is an optional element of life cycle assessment, which is a means to relate the impact category indicators (e.g. kg CO2 equivalent) to some reference. In the case of external normalization this reference group of indicators is outside of the study boundary and is intended to provide context to the characterized impacts and some degree of error identification (ISO 2006b, Heijungs and Guinée 2012, Matthews et al. 2013). In addition to presenting impact category indicator results, this study employs external normalization to the US per capita per day baseline developed by Ryberg et al. (2014).

RESULTS

This environmental data synthesis is divided into three sections according to the life cycle stages of lube oil as, (1) acquisition, (2) certain aspects of use, and (3) disposal by distillation to MDO. Data collection, comparisons among literature sources, and environmental impact results are separately discussed for each stage.

Acquisition of lube oil. Acquisition of lube oil involves exploration for and extraction of crude oil, refining crude oil into base oil, and blending additives into the oil to give it desired properties, after which is becomes finished lubricating oil. Additives typically comprise about 20% of the final product by mass according to Raimondi et al. (2012).

Quantified environmental impacts (or LCI data in some cases) for exploration for crude oil through refining to base oil are reported in a number of sources (Raimondi et al. 2012, Geyer et al. 2013, ANL 2013, Gabi 2014). Impacts from the GaBi 6 extension database (2014) and Geyer et al. (2013) were well aligned, with the only fairly major differences in human health impacts, likely because Geyer et al.'s (2013) data were developed for California and GaBi (2014) for the U.S. in general (it should also be noted that these data were not directly reported in Geyer et al. 2013, but were back-calculated using available information in the report). GREET (ANL 2013) is a tool that gives inventory data for regulated and greenhouse gas emissions only. GWP and SCP derived from GREET data were similar to the previous two sources (since those are driven largely by regulated and greenhouse gas emissions); however the other impact categories showed significantly lower or zero impacts, because they are less driven by regulated and greenhouse gas emissions. Still the agreement on expected impact categories supports the accuracy of Geyer et al. (2013) and the GaBi 6 (2014) database. Results from Riamondi et al. (2012) were similar to Geyer et al. (2013) and GaBi (2014) for GWP, AP, HHNCP, and HHCAP (larger differences in HHCP, ETP, and ODP may be attributable to the use of European data and/or uncertainty introduced in impact methodology conversion). The GaBi 6 database (2013) was chosen as the data source for this study since it includes general

U.S. data, has the most complete inventory data, and was integrated into the GaBi 6 software which reduced the potential for errors in calculation. Due to the supported similarities in impacts from literature sources, it was considered to be a high quality data source.

The GaBi database source is part of a highly reviewed set of petroleum product processes developed by PE International. All effects related to the feedstock (crude oil) were allocated on an energy (calorific value) basis to the various refinery products, while all impacts of refinery processes were allocated on a mass percentage of throughput (GaBi 2014). This allocation procedure allowed impacts to be modelled based on the processes actually applied to base oil production.

Blending is the process of introducing additives into the oil, which include at least detergents, dispersants, viscosity modifiers, and antioxidants. The impacts involved in creating these additive materials are typically not included in LCAs of lubricating oil and could only be found in one literature source (Raimondi et al. 2012). These data are included in this paper, but with caveats because additive profiles can vary significantly between different lube oil formulations, impacts were developed from European data, proxies for the additives were used instead of the actual additives, energy for physical blending processes were not included, and impacts were reported based on percentage of the base oil impacts in each impact category rather than on characterized impact values. Still, using data of this type with estimation or significant assumptions is usually preferable to the alternative of omitting processes that may not be well studied (Klöpffer and Grahl 2014).

Table 1 shows the impacts for creating 3.79 liters (one gallon) of new oil. The column titled “Production of 3.79 L of base oil” is to be used if additive blending is considered too uncertain and assumes that the finished oil product is 100% base oil. The column titled “Additive blending for 3.79 L of lube oil” is the impact potentials of the additive makeup from Raimondi et al. (2014) for 3.79 liters (one gallon) of finished lube oil. The column titled “Total including blending” is the total impacts for 3.79 liters (one gallon) of finished oil assuming 80% of the base oil impacts (since 20% of the finished oil is additives) and all of the additive blending impacts. Therefore, the first column should be used if additives are ignored and the last column should be used if they are considered. Figure 2 shows the impacts (including blending) normalized to the U.S. per day per capita baseline.

Table 1. Lube oil acquisition impacts for 3.79 liters (one gallon) of new oil.[a]

Impact Category	Unit	Production of 3.79 L of base oil[b]	Additive blending for 3.79 L of lube oil[c]	**Total including blending[d]**
Global Warming	kg CO_2 eq	4.56	2.05	**5.70**
Acidification	mol H^+ eq	1.20	0.295	**1.25**
Eutrophication	kg N eq	7.73×10^{-4}	4.65×10^{-5}	$\mathbf{6.65\times10^{-4}}$
Ecotoxicity	CTUeco	1.81	0.143	**1.59**
Human Health Cancer	cases	4.50×10^{-9}	6.98×10^{-9}	$\mathbf{1.06\times10^{-8}}$
Human Health Non-Cancer	cases	4.03×10^{-7}	1.13×10^{-7}	$\mathbf{4.36\times10^{-7}}$
Human Health Criteria Air	kg PM_{10} eq	3.80×10^{-3}	9.59×10^{-4}	$\mathbf{4.00\times10^{-3}}$
Smog Creation	kg O_3 eq	0.250	No Data	**2.00**
Ozone Depletion	kg CFC-11 eq	2.48×10^{-10}	2.45×10^{-11}	$\mathbf{2.23\times10^{-10}}$

[a]Langfitt and Haselbach 2014;
[b]GaBi (2014);
[c]Raimondi et al. 2012; [1]Langfitt and Haselbach 2014;
[d]0.8*[Base Oil] for base oil impacts since ~20% is additives in finished lube oil

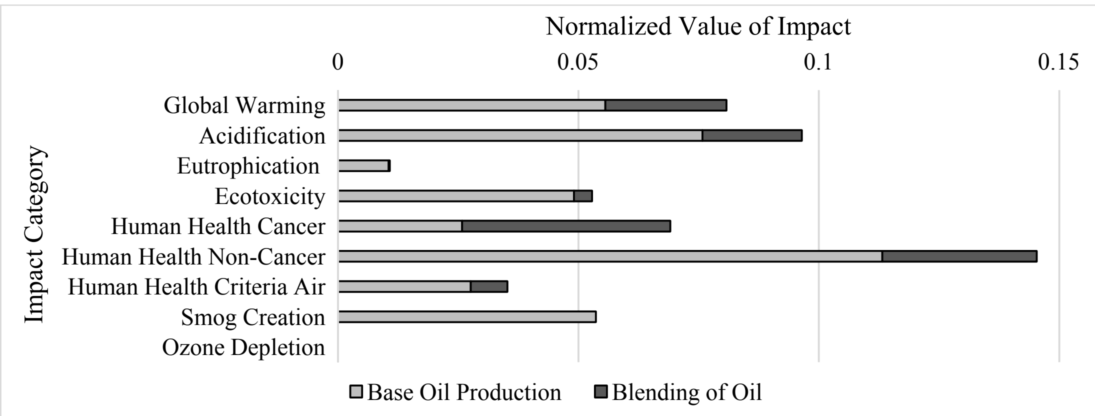

Figure 2. Lube oil acquisition impacts for 3.79 liters (one gallon) of lube oil externally normalized to total US per capita per day impacts (blending ratio impacts from Raimondi et al. 2012, US normalization references from Ryberg et al. 2014 and U.S. population from 2010 census).

Use of lube oil (combustion). Lube oil can be introduced to engines in a number of ways based on the engine configuration and type. In some systems, oil is nearly conservative (not used during operation, but rather cycled through a closed system) while in others it can be virtually total loss, or somewhere in between (Christensen 2010). For instance, based on top-up records provided from a large ferry fleet, some vessels could burn hundreds of gallons of oil in a month while others may burn well under a tenth of that. This section therefore covers only the 'use' of the oil as in being combusted during the use phase. (It should be noted that any oil combusted in the engine and accounted for as "used" no longer requires disposal by another method, so double counting should be avoided here.)

Simple combustion modelling was employed to account for emissions from the combustion of the lube oil in the engine. This model provided emissions of carbon dioxide, sulfur dioxide, hydrogen chloride, nitrogen oxides, and dust based on average used oil composition. For metals emissions, the model suggests that nearly all of the metals in the oil are carried out in the emissions. Due to this model, cut-off criteria were defined as any substances not previously mentioned (where metals included were all of those analyzed in the oil analysis). The quantities of emissions were modeled in GaBi 6 using TRACI 2.0, in addition to the average oil metals concentrations from one large ferry vessel determined through oil analysis. Although lubricating oil has energy value, this study assumes that much less oil is burned than fuel in these engines and that there is negligible offset of fuel needs based on oil burning even though in some engines this may not be true. Additionally, emissions could vary considerably based on the composition of the oil (particularly concentrations of metals such as zinc and copper, which can heavily impact ETP). Another assumption is that there are not emissions control systems removing these types of materials from the exhaust stream of the vessel.

Leaking of oil from the engine could be included in this stage, and if significant leakage were common that could be an important consideration. However, discussion with experts has indicated that during normal usage in marine vessels there is very little or no leaking of oil, so leaked oil is not considered in this environmental data synthesis. Characterized impacts of the use 'combusted' stage are shown in Table 2, while normalized impacts are shown in Figure 3.

Table 2. Impacts for combustion of 3.79 liters (one gallon) of lube oil.

Impact Category	Unit	Audibert 2006 ("WSF" oil)
		TRACI 2.0
Global Warming	kg CO_2 eq	9.59
Acidification	mol H^+ eq	2.25
Eutrophication	kg N eq	7.69×10^{-4}
Ecotoxicity	CTUeco	1.09
Human Health Cancer	cases	2.42×10^{-10}
Human Health Non-Cancer	cases	5.20×10^{-7}
Human Health Criteria Air	kg PM_{10} eq	5.93×10^{-3}
Smog Creation	kg O_3 eq	0.343
Ozone Depletion	kg CFC-11 eq	0

Note: Audibert did not complete impact category assessment. Their emission data were converted to LCA impacts by the authors. In addition, the metals were based on oil analysis data provided by Washington State Ferries (WSF).

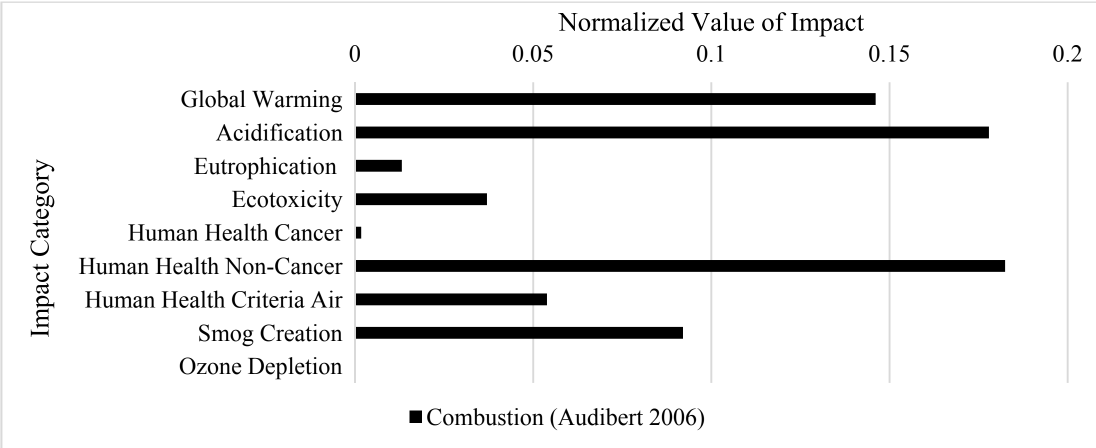

Figure 3. Lube oil use (combustion) impacts for on3.79 liters (one gallon) of lube oil externally normalized to total US per capita per day impacts (US normalization references from Ryberg et al. 2014 and U.S. population from 2010 census).

Disposal of lube oil. Combustion for energy recovery, re-refining into fresh base oil, and distillation to marine diesel oil (MDO) are three of the most common disposal approaches for used lube oil. This report focuses on disposal by distillation to MDO, although disposal by combustion could be considered similar to the previous impacts from use (but including offset for avoided fuel use), and information on impacts of re-refining are included in Geyer et al. (2013).

Inventory data was not available for distillation of lube oil to MDO, so literature LCA data had to be used instead. Few studies address this topic; however there is one that provides a simplified model for the distillation process which includes energy and major chemical inputs (Boughton and Horvath 2004) and another which used more detailed models, but only reported characterized impact results from their LCA, with no detailed data on the inputs and outputs of the processes (Geyer et al. 2013). The results of the two studies were relatively similar with the exception of acidification and ecotoxicity, which were both much higher in Geyer et al. (2013). This is likely because the latter was significantly more detailed in its modeling and data inventory. Because of this and the fact that the study was more up to date, Geyer et al. (2013) was chosen as the data source for the disposal process. Data for that study were collected with a base year of 2010, a focus on California, and came from both primary and secondary sources, with relatively high data quality. Further information on system boundaries, cut-off criteria, allocation, data quality, and other methodological decisions are discussed in Geyer et al. (2013).

When lube oil is distilled, it produces at least two usable products, marine diesel/distillate fuel and asphalt additive. According to Geyer et al. (2013), when 1 kg of used oil is reprocessed through this method, 0.50 kg of fuel, 0.31 kg of asphalt additive, and 0.19 kg of light ends, sludge, and other wastes are produced. Usable products produced in the disposal of used oil displace some needed to produce those products from other sources. Accordingly, it is appropriate to consider that 0.50 kg of fuel and 0.31 kg of asphalt additives do not need to be produced through another method, and the impacts that would have resulted from their production are discounted from the used oil disposal process. Following the same assumptions as Geyer et al. (2013), the displaced fuel is assumed to be no. 2 distillate fuel and the displaced asphalt product to be bitumen. Table 3 shows the potential impacts for distillation of used oil to MDO, offset of primary fuel production, and total impacts. Notice that nearly all impact categories have negative total impacts. This simply means that creating MDO from used lubricating oil has fewer impacts than producing it from crude oil. Figure 4 shows the normalized results for disposal.

Table 3. Lube oil disposal impacts for 3.79 liters (one gallon) of used oil.

Impact Category	Unit	Distillation of used oil to MDO[a]	Offset primary products[a]	**Total including offset**
Global Warming	kg CO_2 eq	0.47	-1.33	-0.87
Acidification	mol H^+ eq	0.20	-0.466	-0.27
Eutrophication	kg N eq	8.66×10^{-5}	-2.23×10^{-4}	-1.37×10^{-4}
Ecotoxicity	CTUeco	0.53	-0.533	-0.003
Human Health Cancer	cases	3.70×10^{-10}	-6.20×10^{-10}	-2.50×10^{-10}
Human Health Non-Cancer	cases	2.57×10^{-8}	-6.96×10^{-8}	-4.39×10^{-8}
Human Health Criteria Air	kg PM_{10} eq	5.96×10^{-4}	-1.65×10^{-3}	-1.05×10^{-3}
Smog Creation	kg O_3 eq	0.007	-0.089	-0.082
Ozone Depletion	kg CFC-11 eq	1.33×10^{-9}	No Data	1.33×10^{-9}

[a]Geyer et al. 2014;
Note: Ozone depletion for distillation to MDO derived from Boughton and Horvath (2004).

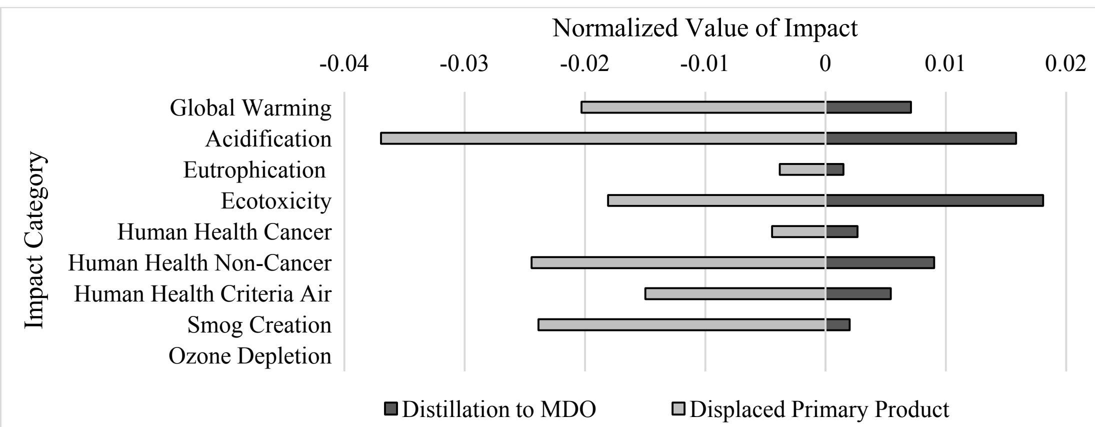

Figure 4. Lube oil disposal impacts for 3.79 liters (one gallon) of used oil externally normalized to total US per capita per day impacts (distillation and primary product offset impacts from Geyer et al. 2014, US normalization references from Ryberg et al. 2014 and U.S. population from 2010 census).

CONCLUSION

These results provide marine industry decision-makers with additional information when considering potential environmental impacts in decisions that might alter lube oil use. Economic, social, and feasibility considerations are not included in this environmental analysis, but should also factor into decisions. The most significant limitations of this study were that most data are not specifically for marine vessels and that composition of oil can vary greatly and have a large impact on results. Still, as environmental goals are increasingly a part of public agency and private business motivations, the ability to consider applicable environmental data is very valuable.

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Assessment of Life Cycle Energy Saving and Carbon Reduction of Using Reclaimed Asphalt Concrete

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Abstract

This study uses the Pavement Life-cycle Assessment Tool for Environmental and Economic Effects (PaLATE) to study the benefits of energy savings and carbon emission reduction derived from using Reclaimed Asphalt Pavement (RAP). According to the findings, producing RAP mixture with 30% of reclaimed asphalt materials results in 16% of energy saving and 20% of carbon emission reduction while compared to that of producing the same amount of virgin hot mix asphalt mixture. The mitigation of energy consumption and carbon emission is mainly due to the saving on virgin asphalt binder. However, it is found from a life cycle approach, only if RAP pavement gives a service life at least 80% of that of virgin materials, using RAP becomes a feasible way to make the pavement greener.

INTRODUCTION

Reclaimed asphalt pavement (RAP) has been considered a good recycled material from both economic and environmental perspectives because of its low cost and wide availability (Aravind and Das, 2007). After proper treatment, it can be reused by mixing with specific amount of virgin binder, recycle agents (optional) and aggregates. Many studies claimed that using RAP asphalt concrete causes less environmental impact than using virgin asphalt concrete does, since adding RAP in hot asphalt mixtures (HMA) reduces the usage of natural materials (Chiu et al., 2008; Lin, 2010; Chowdhury et al., 2010). Nevertheless, most studies often neglect that producing RAP asphalt mixtures also requires extra energy, and the performance of RAP mixture may be different from that of virgin asphalt concrete (Chiang, 2002; Aguiar-Moya et al., 2011; Nash et al., 2012).

As the process of producing HMA mixtures showed, extra crushing and sieving are necessary in preparing the RAP for reuse, and the odor removal is a requirement under environmental regulations in some countries (Figure 1). With these additional processes, it is likely that producing RAP mixtures may result in more environmental impact than some previous studies showed.

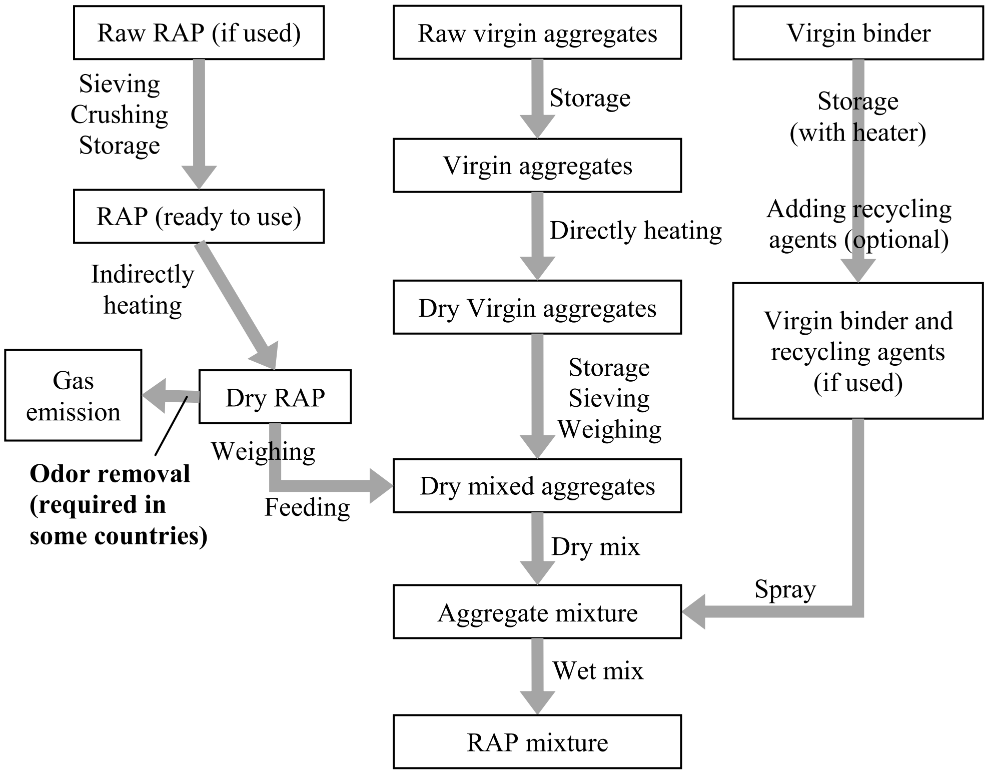

Figure 1. Material flow diagram of producing HMA mixture.

In addition, how the performance of RAP asphalt concrete compares with the virgin asphalt concrete is still not very clear. Since the mixture becomes stiffer with increasing RAP content (Apeagyei et al., 2011; Attia and Abdelrahman, 2010; Behnia et al., 2010; Hajj et al., 2012), some studies claim that the performance of RAP asphalt concrete is equal to or even better than that of the virgin asphalt concrete because its resistance to permanent deformations, such as rutting and roughness, is improved (West et al., 2009; Maupin et al., 2009; Nash et al., 2012). However, other studies showed different results. One study indicated that the service lives of test sections with RAP are shorter than that of the sections without RAP, because cracking and potholes occur more easily (Chiang, 2002). Another study compared over 10 years of data obtained from the Long-Term Pavement Performance database of several pavement sections in Texas (Aguiar-Moya et al., 2011) and implied that there is no reason to support using RAP in pavement construction, since the service life of RAP overlays is 30% to 60% shorter than that of the virgin overlays. The shorter the service life of pavements, the more maintenance is required in a fixed analysis period. Therefore, when assessing the environmental impact of a pavement construction from the life cycle approach, it is important to take the factor of performance, in terms of length of the pavement's service life, into consideration.

OBJECTIVE AND METHODOLOGY

The aim of this study is to investigate whether using RAP in pavement construction is feasible and effective in saving energy and reducing carbon emissions.

Both the cradle-to-gate and life-cycle environmental benefits of using RAP asphalt concrete are evaluated.

Firstly, the energy consumption and carbon emission of producing a specific amount of HMA mixtures with different RAP content levels are calculated by using the Pavement Life-cycle Assessment Tool for Environmental and Economic Effects (PaLATE), and then the effects due to the extra processes required for producing RAP asphalt mixtures and the energy saving strategies in the plants are also discussed.

Secondly, the environmental benefit of using RAP asphalt concrete is inspected from the life cycle approach. This study expresses the difference on performance between the virgin asphalt concrete and the RAP asphalt concrete with varied RAP content levels (10%, 20%, 30%, and 40%) by the ratio of the lengths of their service lives, and then examines the corresponding life cycle energy consumption ratios and life cycle carbon footprint ratios to represent the environmental advantage of using RAP asphalt concrete.

ANALYSIS TOOL AND DATA

The PaLATE. This study uses PaLATE (Horvath 2004) as the source of the carbon emission data of each material. PaLATE is a free-of-charge database that assesses the life cycle environment impact of pavement construction. This database was developed by the Consortium on Green Design and Manufacturing and the Recycled Materials Resource Center at University of California at Berkeley. Many studies recommended using the PaLATE to inventory the environmental impact as a decision making support tool (Cui and Zhu, 2011; Chan et al., 2011; Podborochynski et al., 2012).

PaLATE includes a detailed database built on literature review, which includes the environmental impact and cost information for several types of materials and processes commonly used in pavement construction. It provides information of several recycled materials and on-site recycling processes which is not provided by other life cycle assessment tools. Table 1 lists the information of some pavement materials and the plant process used in this study. Since PaLATE considers RAP as a type of recycled materials, the amounts of energy needed and carbon dioxide emitted due to RAP preparation are both assumed as zero. This assumption is not a true statement, since the raw RAP obtained from milling must be crushed and sieved before it can be used. According to a study in Taiwan (Lin, 2010), it costs 22.857 kWh electricity and 0.03 liters of diesel to prepare each ton of RAP. Although the energy consumption and carbon emission of the RAP preparation are relatively low compared to the other parts of materials, it should not be ignored. Therefore, this study includes this into consideration together with other information of PaLATE.

Table 1. Environmental impact of asphalt pavement mixing.

Items	*Description*	*Energy consumption (MJ/ton)*	*Carbon emission (kg-eCO$_2$/ton)*
Aggregates	Prepare per ton of material	154	10.92
Asphalt binder	Prepare per ton of material	19,757	1,122
RAP*	Prepare per ton of material	87.54	1.56
Batch plant process	Produce per ton of the mixture	227	16.80

*converted based on information provided by plants in Taiwan (Lin, 2010).

The inputs to PaLATE are the pavement structure, material compositions, and construction processes. The user only need to input the amount and type of materials, construction activities, and transportation used in the project, and then the environmental impact, such as carbon emission and energy consumption, will be summed up according to the information in the database of PaLATE (Nathman et al. 2009). This feature makes PaLATE flexible for various types of studies.

Material compositions. Lin (2010) compiled the cradle-to-gate carbon emission of an HMA batch plant in Taiwan to evaluate the respective carbon footprints of producing virgin HMA and RAP asphalt mixtures. In that study, the emissions from transporting the materials were also calculated based on the average values of historical data. The result showed that producing 30% RAP mixture per ton emits about 85% of carbon generated of producing virgin mixture. However, the scope of inventory in that study does not include the savings on virgin binder, so the amount of carbon emission of producing RAP mixtures is overestimated.

In fact, the higher the RAP content in the mixture, the more virgin materials can be preserved. FHWA provided the following equation for estimating the amount of virgin binder needed when manufacturing HMA with RAP (Kandhal and Mallick, 1997):

$$P_{nb} = P_b - \frac{(100 - r)P_{sb}}{100} \tag{1}$$

Where,

P_{nb} is the asphalt content of the RAP mixture, as percentage (%) in total weight,

r is the percentage of virgin aggregate, as percentage (%) in weight of total aggregates,

P_b is the asphalt content, the amount of binder needed in virgin mixture, and

P_{sb} is the asphalt content in RAP (plus the weight of recycling agents if used).

According to the experience of Taiwanese HMA plants, when the asphalt content (P_b) is 5% and the RAP content is 30%, the amount of virgin binder (P_{nb}) should be 3.5%. By using equation (1), the asphalt content in the RAP (P_{sb}) is 4.7%, which is similar to that reported by Aguiar-Moya et al. (2011). Therefore, the asphalt content in RAP is assumed as 4.7% of total weight, and the amount of virgin binder needed under different RAP contents can be calculated using equation (1).

The default densities of virgin aggregate, asphalt binder, and RAP are 1.64 ton/m^3, 1.10 ton/m^3, and 2.42 ton/m^3, respectively, in the PaLATE database. Because the density of each mixture varies with the material composition, and a fixed volume of mixtures will be needed for a specific construction project, this study calculates the material compositions per unit volume of mixture at each RAP content level as shown in Table 2.

Table 2. Material composition with different rap content levels.

RAP Content (%)	*Density (ton/m^3)*	*Materials*					
		RAP		*Virgin Binder*		*Virgin Aggregate*	
		Volume (m^3)	*Weight (ton)*	*Volume (m^3)*	*Weight (ton)*	*Volume (m^3)*	*Weight (ton)*
0	1.60	0.000	0.000	0.072	0.080	0.928	1.518
10	1.65	0.068	0.165	0.068	0.074	0.862	1.410
20	1.71	0.142	0.343	0.062	0.069	0.797	1.303
30	1.78	0.221	0.534	0.057	0.062	0.723	1.184
40	1.85	0.305	0.738	0.050	0.055	0.643	1.052

CRADLE-TO-GATE ENVIRONMENTAL BENEFITS OF USING RAP

Effect of RAP content levels. The cradle-to-gate processes of HMA include manufacturing virgin aggregates, transporting aggregates to the HMA plants, producing virgin binder, transporting the asphalt binder from the refinery to the HMA plant, and applying the materials to hot mix process. This study assumed that there are sufficient materials stored in the plants, and only focuses on the calculation of energy consumption and carbon emissions of the manufacture of materials and the hot mix process. Equations (2) and (3) show how the environmental impacts of energy consumption and carbon emission are calculated in this study. Equations (4) and (5) are used to estimate the environmental benefits of adding various percentages of RAP in HMA mixtures.

$$EC_{i\%} = EC_{i\%,\,VA} + EC_{i\%,\,VB} + EC_{i\%,\,PP} \tag{2}$$

$$C_{i\%} = C_{i\%,\,VA} + C_{i\%,\,VB} + C_{i\%,\,PP} \tag{3}$$

$$\text{Energy Saving}\,(\%) = \frac{(EC_{0\%} - EC_{i\%})}{EC_{0\%}} \tag{4}$$

$$\text{Carbon Reduction}(\%) = \frac{(C_{0\%} - C_{i\%})}{C_{0\%}} \tag{5}$$

Where,

$EC_{i\%}$ is the Energy consumption of mixture with i% RAP content (MJ/m^3),

$C_{i\%}$ is the carbon emission of mixture with i% RAP content (kg-eCO$_2$/m^3),
VA means Virgin Aggregate,
VB means Virgin Binder, and
PP means Plant Process, which includes the hot-mix process and the RAP preparation.

The calculated results are shown in Table 3, and the higher the RAP content, the higher environmental benefits can be obtained. When the mixture contains 30% of RAP, producing RAP mixtures saves 13.75% of energy and reduces 19.70% of carbon emission compare to that of producing the virgin mixture.

Table 3. Environmental benefit of RAP mixtures.

RAP Content (%)	*Energy Consumption (MJ/m^3)*	*Energy Saving (%)*	*Carbon Emission (kg-eCO$_2$/ m^3)*	*Carbon Reduction (%)*
0	2161.20	0.00	108.71	0.00
10	2071.72	4.14	102.31	5.89
20	1975.42	8.60	95.24	12.39
30	1863.95	13.75	87.29	19.71
40	1737.47	19.61	78.46	27.82

The energy saving and carbon reduction of producing mixtures with RAP can be further sub-divided into three major categories. Table 4 shows the percentage of energy saved and carbon emission reduced from aggregate, asphalt binder, as well as plant process. It is obvious that the environmental benefit of using RAP is mainly contributed by savings on asphalt binder. On the contrary, there is an increase in environmental impacts caused by the plant process of producing RAP asphalt mixture. This is primarily due to the extra processes, such as crushing and sieving, needed to produce RAP mixtures.

Table 4. Energy saving and carbon emission reduction from aggregate, asphalt binder, and plant process.

RAP	*Aggregate*		*Asphalt Binder*		*Plant Process**	
Content (%)	*Energy Saved (MJ/m^3)*	*%*	*Energy Saved (MJ/m^3)*	*%*	*Energy Saved (MJ/m^3)*	*%*
10	16.70	18.66	98.14	109.67	-25.35	-28.33
20	33.18	17.86	209.54	112.79	-56.94	-30.65
30	51.61	17.36	334.20	112.43	-88.55	-29.79
40	71.88	16.96	472.12	111.42	-120.26	-28.38
RAP	Aggregate		Binder		Plant Process	
Content (%)	Carbon Reduced (kg-eCO$_2$/m^3)	%	Carbon Reduced (kg-eCO$_2$/m^3)	%	Carbon Reduced (kg-eCO$_2$/m^3)	%
10	1.18	18.48	5.57	87.08	-0.36	-5.55
20	2.35	17.44	11.90	88.33	-0.78	-5.77
30	3.65	17.06	18.98	88.59	-1.21	-5.65
40	5.09	16.83	26.81	88.64	-1.65	-5.46

*Includes the environmental impact due to RAP preparation.

Energy consumption and carbon emission during asphalt binder preparation. Some past studies debated whether the environmental impacts of producing virgin binder should be included when calculating the cradle-to-gate environmental impacts of HMA mixtures. Table 5 lists the energy consumption and carbon emission of the virgin mixture (0% RAP) and 30% RAP mixtures. It is clear that the manufacture of asphalt binder leads to the greatest environmental impact. In the virgin mixture case, the energy consumption of binder manufacture comprises more than 70% of the total value, and its carbon emission comprises more than 80% of the total value.

Table 5. Energy consumption and carbon emission of HMA manufacture.

	Energy Consumption		*Carbon Emission*	
	MJ/m^3	*%*	*kg-eCO$_2$/m^3*	*%*
RAP content 0% (virgin)				
Aggregate	234.15	10.83	16.58	15.25
Asphalt Binder	1564.89	72.41	88.87	81.75
Batch Plant Process	362.16	16.76	3.26	3.00
Total	2161.20	100.00	108.71	100.00
RAP content 30%				
Aggregate	182.55	9.79	12.93	14.81
Asphalt Binder	1230.69	66.03	69.89	80.07
Batch Plant Process	403.97	21.67	3.64	4.17
RAP Preparation	46.74	2.51	0.83	0.95
Total	1863.95	100.00	87.29	100.00

As shown in Table 6, if the manufacture of asphalt binder is not included in the evaluation, both the benefits on energy saving and carbon reduction decreased considerably. Although asphalt binder is a byproduct of oil manufacture process, the energy spent and carbon emissions are still taken into consideration when evaluating the environmental impact of asphalt mixtures in this study.

Table 6. Environmental benefits ot considering binder preparation or not.

	Energy Saving (%)		*Carbon Reduction (%)*	
RAP Content (%)	*Consider the asphalt binder preparation*	*Not consider the asphalt binder preparation*	*Consider the asphalt binder preparation*	*Not consider the asphalt binder preparation*
10%	4.14	-1.45	5.89	4.17
20%	8.60	-3.98	12.39	7.92
30%	15.92	-6.19	20.47	12.32
40%	19.61	-8.11	27.82	17.32

Energy saving strategies in hot mix asphalt plants. As mentioned previously, the manufacturing processes for RAP mixtures differ slightly from the virgin mixtures due to its odor removal during RAP process. In Taiwan and some Asian countries, all RAP mixture plants must use some measures to remove odor from the emitted gas.

This odor-removal equipment is commonly a heating tower which removes the odor by burning the odor gas with high-temperature flames at approximately 600 to 700℃. According to records provided by the local HMA plants, the fuel consumption of the odor-removal tower is about 2.5 liters of heavy oil per ton for 30% RAP mixture produced. This amount of heavy oil consumption can be converted to 101.365 MJ of energy (Taiwan Bureau of Energy, 2011). For each cubic meter of 30% RAP asphalt concrete, the process of odor removal requires 180.35MJ of energy. However, PaLATE does not account for emissions from odor removal. Because of this odor removal process, the environmental benefit of using RAP is lowered. Table 7 shows the energy savings of using RAP decreases significantly from 13.75% to 5.41% when the odor removal process is accounted for directly.

Table 7. Energy saving of using 30% RAP along different energy saving efficiency in HMA plants.

		Energy consumption (MJ/m^3)	*Energy savings (%)*
Without odor removal			
0% RAP content (virgin mixture)		2161.20	
30% RAP content		1863.95	13.75
With odor removal			
Energy Saving Efficiency (by reusing the heat of odor removal)	0%	2044.38	5.41
	5%	1942.16	10.14
	10%	1839.94	14.86
	15%	1533.29	29.05

To reduce energy cost, many HMA plants have implemented energy saving strategies by reuse the heat during odor removal. The temperature of emission gas is about 300 to 400°C at the odor-removal tower , and this heat can be used to keep the ready-mix asphalt warm. Some plants use this heat to dry the virgin aggregates and eliminate the odor at the same time.

The benefit of energy savings through the use of RAP can be raised with an increase in energy reuse efficiency. As shown in Table 7, when a plant reduces total energy consumption by 10%, the energy consumption of producing the 30% RAP mixture is 14.86% less than that of producing the same amount of virgin mixture, which is similar to the energy savings when not considering odor removal. Reusing the heat not only saves money for energy but also compensates the underestimation of RAP asphalt concrete's environmental impact in PaLATE.

EFFECTS OF PERFORMANCE

The service life of pavement determines the number of major maintenance cycles needed in the analysis period. It is clear that more maintenance leads to more material usage as well as higher levels of energy consumption and carbon emission. According to the analysis results in this study, producing RAP mixtures indeed consumes less energy and generates less carbon emissions than producing the same amount of virgin mixtures even with none energy reused during odor removal process. However, the performance of RAP should be taken into consideration in order to understand the true saving of energy and carbon dioxide of using RAP. This is because that the performance of RAP will affect the maintenance cycle and consequently influence the comparison results.

There is no reliable performance prediction tool which can predict the performance of pavement containing RAP so far. The latest pavement design guide, MEPDG, published by American Association of State Highway and Transportation Officials (AASHTO) developed pavement distress models based on historical data and mechanistic analysis (AASHTO, 2002). However, the design software associated with MEPDG does not have the functionality to predict the service life of pavements containing RAP. Since the use of RAP often makes mixtures stiffer, the RAP mixtures are likely to have longer predicted service lives than those of virgin mixtures by MEPDG if a higher viscosity value is used (Praticò et al. 2011; Podborochynski et al. 2012). Accordingly, it is not appropriate to use MEPDG for predicting the service life of pavement containing RAP.

To overcome the problem, this study uses Service Life Ratio (SLR) of RAP concretes to the virgin concrete to express their difference in performance instead. Assuming that the thickness of the surface layer does not change with the various designs of using mixture with different RAP content, the required number of major maintenance events (usually mill and overlay) vary with the RAP content if it's corresponding performance changes. The life cycle energy saving and the life cycle carbon reduction of RAP concrete to the virgin mixture can be simplified and described as Equations (6) and (7), respectively.

$$LCES_{i\%} = 1-(MR_{i\%} \times ER_{i\%}) = 1-\frac{ER_{i\%}}{SLR_{i\%}} \quad (6)$$

$$LCCR_{i\%} = 1-(MR_{i\%} \times CR_{i\%}) = 1-\frac{CR_{i\%}}{SLR_{i\%}} \quad (7)$$

Where,

$LCES_{i\%}$ is the life cycle energy saving of i% RAP mixtures relative to the virgin mixture,

$LCCR_{i\%}$ is the life cycle carbon reduction of i% RAP mixtures relative to the virgin mixture,

$MR_{i\%}$ is the ratio of the number of required major maintenance events for pavement containing i% RAP to that of the virgin pavement,

$ER_{i\%}$: is the energy consumption ratio of per unit volume of i% RAP mixture to the virgin mixture,

$CR_{i\%}$ is the carbon emission ratio of per unit volume of i% RAP mixture to the virgin mixture,

$SLR_{i\%}$ is the service life ratio of pavement containing i% RAP to that of virgin pavement, which is the reciprocal of $MR_{i\%}$.

The $ER_{i\%}$ and $CR_{i\%}$ values of HMA with different RAP content levels are shown in Table 8. By applying Eq. (6) and Eq. (7), Figures 2 and 3 illustrate the life cycle energy saving and the life cycle carbon reduction of RAP mixture with different RAP contents. Only when the LCES or LCCR value is positive, the use of RAP in the HMA mixture has environmental benefit.

Table 8. $ER_{i\%}$ and $CR_{i\%}$ values of HMA with different RAP content levels.

RAP content	*Energy Consumption* (MJ/m^3)*	*$ER_{i\%}$*	*Carbon Emission* ($kg\text{-}eCO_2/m^3$)*	*$CR_{i\%}$*
10%	2111.1898	0.9599	104.8528	0.9431
20%	2016.3270	0.9167	97.8738	0.8804
30%	1906.5286	0.8668	90.0295	0.8098
40%	1781.7236	0.8101	81.3125	0.7314

*Includes the environmental impact of a 50-km-long transportation from the plant to the site.

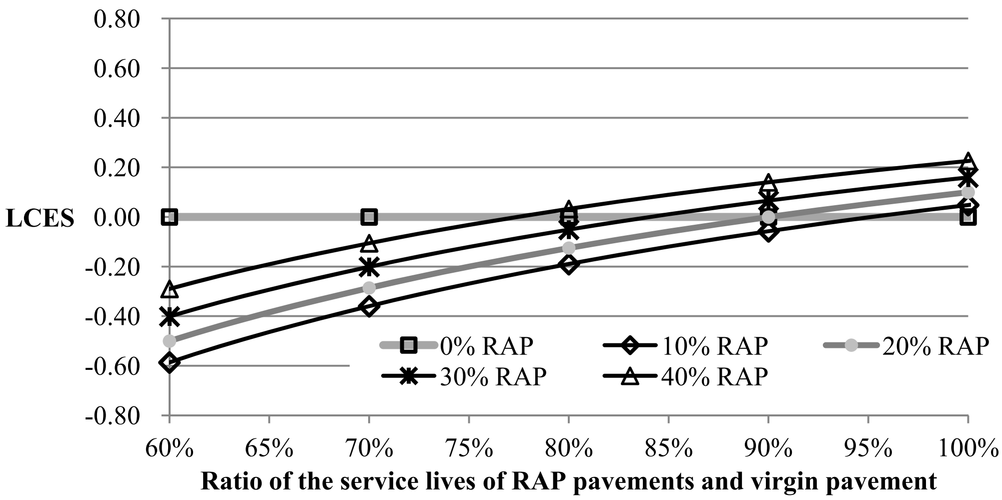

Figure 2. Life cycle energy saving of using RAP mixtures.

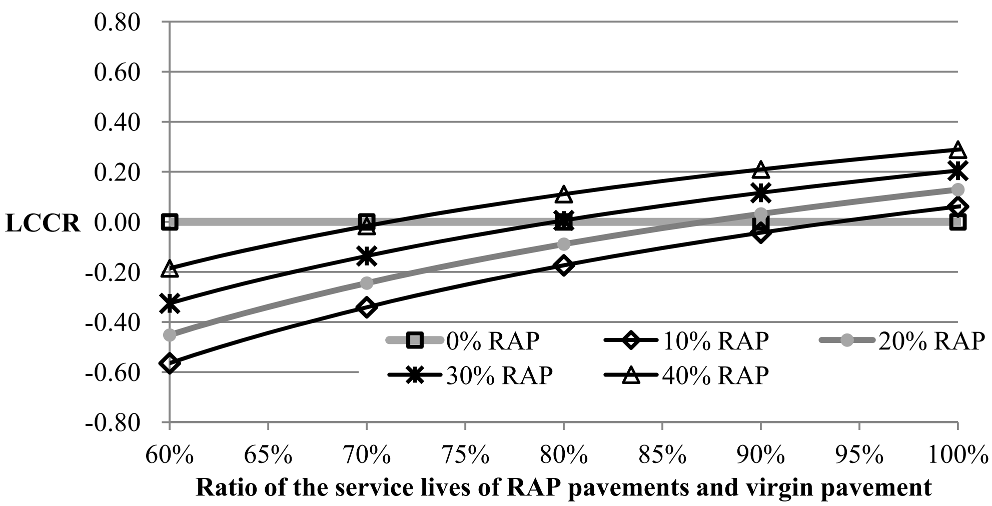

Figure 3. Life cycle carbon reduction of using RAP mixtures.

Increasing RAP content leads to lower carbon emission. The higher the RAP content in the mixture, the lower the SLR needed to achieve positive environmental benefits. The study results show that once the SLR of the 40% RAP concrete is 81% or more, it will be more sustainable than pavement with virgin asphalt mixture in terms of energy consumption. However, if the RAP content is only 10%, the performance of RAP concrete must be at least 96% of that of the virgin concrete to make the pavement greener. The performance threshold to reduce carbon emission is lower than that of saving energy. For RAP concretes with 40% RAP content, as long as the SLR reaches 73%, it emits less carbon during the analysis period. As to the 10% RAP content case, its SLR must be up to 94% in order to have the advantage.

The study shows that from the life cycle approach, using RAP in HMA has a great potential to save energy and to reduce carbon emission. However, length of service life of the RAP asphalt concrete is the critical factor in order to achieve the benefits.

CONCLUSIONS

To evaluate whether using RAP asphalt concrete is a feasible and effective way to save energy and reduce carbon emissions, this study used the software program PaLATE to inspect the environmental benefit by considering both cradle-to-gate analysis and life cycle approach.

Although the asphalt binder is a byproduct of petroleum refineries, environmental impacts due to the preparation of binder should be considered when calculating the energy consumption and carbon emission of HMA concrete production. According to the analysis results, producing the same volume of RAP asphalt mixtures indeed causes less environmental impact than producing virgin asphalt mixtures. Adding 30% of RAP into the HMA saves about 14% energy and reduces 20% carbon emission. The higher the RAP content, the greater environmental benefits can be obtained from material production. The mitigation of energy consumption and carbon emission is mainly due to the saving on virgin asphalt binder.

From the life cycle approach, RAP asphalt concrete has the potential to make the pavement construction more environmentally friendly. However, its length of service life is critical to the quantity of benefits. The lower the RAP content, the longer its service life is required in order to gain the environmental benefits.

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Life-Cycle Performance of Concrete Bridge Decks Exposed to Deicer Environments: A New Risk Rating Method

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Abstract

One way to assess the life-cycle performance of bridge decks is to use risk rating factors. Risk rating factors tend to increase with time, as a bridge deck ages and deteriorates. The major influential factors of bridge deck deterioration in cold climates include materials characteristics, construction quality, freeze/thaw cycles, deicer exposure, and overload trucks. Unlike laboratory testing where all test samples are subjected to similar environmental conditions, the concrete samples cored from bridge decks experienced much more complicated conditions. As such, it is a challenge to evaluate the field conditions of concrete structures, pavements and bridge decks through the analysis of their core samples. An assessment tool to facilitate the condition assessment of concrete bridge deck using a limited number of core samples is useful for bridge owners to implement preservation, repair and rehabilitation strategies in a timely fashion. In this study, the maintenance history and gas permeability and strength testing results from limited core samples were analyzed using a simplistic empirical-mechanistic model. The model was developed to assess the current condition of bridge decks exposed to deicer environments. This was made possible by coupling the percolation theory with the power law for strength and permeability. Specifically, the model accommodated input parameters of freeze/thaw cycles, average daily traffic (ADT), average annual deicer usage, and engineering properties measured from deck core samples (e.g., splitting tensile strength and gas permeability coefficient). Microscopic characterization through scanning electron microscopy (SEM) confirmed the validity of the new tool, as it better captured internal damage in concrete decks that showed little signs of surface distress.

INTRODUCTION

One way to assess the life-cycle performance of bridge decks is to use risk rating factors (Stewart, 2001; Wang, 2011a and 2011b, Wenzel, 2013). National Cooperative Highway Research Program (NCHRP) Report 438: *Bridge Life Cycle Cost Analysis* provided an in-depth analysis of life cycle cost for bridges with a detailed quantitative treatment of uncertainty and a project level perspective. In this context, it is necessary to develop a simple model to provide an appropriate risk rating factor which can be employed to quantitatively predict the service life of bridge decks exposed to deicer environments.

Risk rating factors tend to increase with time, as the bridge deck ages and deteriorate (Bu, 2014). The major factors of bridge deck deterioration in cold climates include material characteristics, construction quality, freeze/thaw (F/T) cycles, deicer exposure, and overload trucks (Gode, 2014). The focus of this study is placed on the severity of concrete damage caused by deicer exposure. The use of chemical deicers in cold regions has raised concerns over their potential negative effects on the performance and durability of concrete infrastructure (Pigeon and Pleau, 1995; Shi et al., 2013 & 2014). The application of chemicals is one of the effective methods for snow and ice control on airfields and on roadways during wintery weather (Strong et al., 2010; Ye et al., 2014). Deicers may also pose detrimental effects on concrete through their reactions with cement paste and/or aggregates and thus reduce the integrity and strength of the concrete (Sutter et al., 2008; Shi, 2008; Shi et al., 2010 & 2011).

Unlike the laboratory testing, in which all tested samples are subjected to similar environmental conditions, the concrete samples cored from the field decks experienced much more complicated conditions. As such, it is a challenge to evaluate the field conditions of concrete structures, pavements and bridge decks through the analysis of their core samples. The following two facts further complicate the analysis. First, it is not realistic to obtain a sufficient number of core samples to enable a comprehensive testing program. Second, the core samples may not statistically represent the overall field concrete which is spatially variable in nature.

Fig. 1 presents schematic illustration of a concrete pavement or structure in a small area but with high variability in external environmental conditions. Surface scaling and/or rutting may lead to the formation of some small reservoirs on the concrete. Concrete beneath these small reservoirs (e.g., area A) can accumulate more deicers than the adjacent areas. Therefore, area A and area B would experience different number of F/T cycles and exposure to different deicer concentrations. Note that higher deicer concentration may increase or decrease the number of F/T cycles inside the concrete and thus affect the degree of physical attack during winter, depending on the specific deicer concentration and concrete temperature. Higher deicer concentration generally accelerates the rate of chemical attack by the residual deicer inside the concrete, especially under summer temperatures.

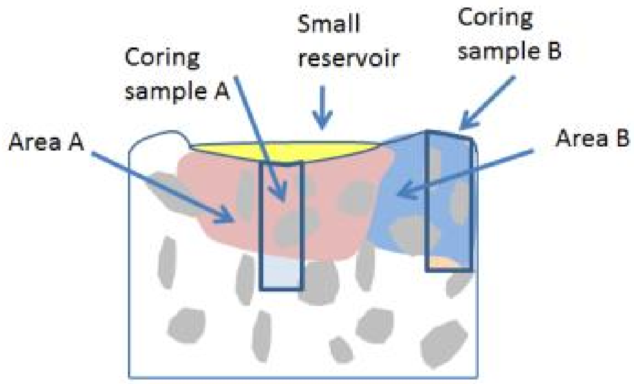

Figure 1. Schematic illustration of field concrete in a small area but with different external environmental conditions.

An assessment tool to facilitate the condition assessment of concrete bridge deck using a limited number of core samples is useful for bridge owners to implement preservation, repair and rehabilitation strategies in a timely fashion. The following sections describe a simple model based on the percolation theory. By combining maintenance history and permeability and strength testing results from deck core samples, the current service condition of bridge decks can be reasonably evaluated.

PREDICTION OF THE SERVICE CONDITION OF THE CURRENT BRIDGES

Gas permeability. According to the percolation theory, the gas permeability of field concrete can be predicted by the following equation (Stauffer, 1979 and 1994):

$$G_c = G_{eff}\left(\frac{p-p_c}{p}\right)^{\tau} \tag{1}$$

where G_{eff} is the effective gas permeability of field concrete, G_c is a constant representing the gas permeability of the new concrete, p is the relative density of the field concrete, and p_c is the porosity threshold that leads to the percolation. The most important parameter here is the critical exponent τ. In the percolation theory, the critical exponent τ and p_c are constants with values of 1.3 and 0.58 in two-dimensional analysis, and 2.0 and 0.16 in three-dimensional analysis.

Mechanical properties. The strength of field concrete (a porous material) as a function of the porosity can also be predicted by a power law, which can be written as:

$$\sigma_{eff} = \sigma_c\left(\frac{\theta-\theta_c}{\theta}\right)^{-\mu} \tag{2}$$

where σ_{eff} is the real strength of field concrete, σ_c is a constant representing the strength of the new concrete, θ is the relative density of the field concrete corresponding to mechanical properties, and θ_c is the porosity threshold that leads to the percolation. A previous study showed that, for the mechanical properties in a percolation system, the value of μ is (Margolina, 1988):

$$\mu = 0.45 \pm 0.06 \tag{3}$$

Coupling the influential factors. In previous studies, the critical exponents of porous media are generally constant variables. However, this universality will be broken if the microstructure of the material does not vary regularly. As a result, it is important to establish the relationship between the critical exponents and the porosity of concrete with exposure to coupling effects, and the porosity of the concrete plays the most important role for the service life prediction. The performance of the concrete can be written as a function of several parameters, including the F/T cycles, mechanical loading history, and deicer attack. Therefore, it can be written as:

$$P = f(F/T)g(\sigma)h(C) \tag{4}$$

where C represents chemical degradation of the concrete due to deicer attack and σ represents physical degradation of the concrete due to externally applied stresses. Assuming the three factors to be independent of each other, Equation (4) can be written as:

$$P = f(F/T)g(\sigma)h(C) = T(service\ life) \tag{5}$$

We assume that all factors follow the power law relationships as follows:

$$f(F/T) = Q_1\left(\frac{\xi - \xi_c}{\xi}\right)^{-f} \tag{6}$$

$$g(\sigma) = Q_2\left(\frac{\xi - \xi_c}{\xi}\right)^{-s} \tag{7}$$

$$h(C) = Q_3\left(\frac{\xi - \xi_c}{\xi}\right)^{-c} \tag{8}$$

Therefore, the service life can be predicted as:

$$P = f(F/T)g(\sigma)h(C) = T(Service life) = P_0\left(\frac{\xi - \xi_c}{\xi}\right)^{(f+s+c)} \tag{9}$$

where f, s, and c represent the critical exponents corresponding to the F/T cycling, external mechanical stress, and chemical attack, respectively. For chemical attack, the degradation is considered as a linear function of deicing chemical quantity. For physical attack, the degradation is considered as a function of F/T cycles and overload truck loadings, which are known to induce cracking of the concrete and increase its porosity.

Equation (9) could be written as:

$$P = P_0 \varphi^{(f+s+c)} \tag{10}$$

where $\varphi = \frac{\xi - \xi_c}{\xi}$, which is defined as the relative density parameter and can be calculated by the average value of $\frac{p - p_c}{p}$ and $\frac{\theta - \theta_c}{\theta}$.

In Equation (10), the values of *f*, *s*, and *c* can be defined as:

$$f = \alpha \frac{F / T_{real}}{F / T_{design}} \tag{11}$$

$$s = \beta (percentage\ of\ the\ overload\ trucks) \tag{12}$$

$$c = \gamma \frac{Real\ quantity}{Designed\ maximum\ quantity} \tag{13}$$

where α, β, and γ represent the relative weights of F/T cycles, fatigue stresses, and chemical attack, which are assumed to be 0.5-0.6, 0.1-0.2, and 0.3-0.4, respectively. These assumptions are made mainly based on field observations. As a result, the rank of the bridge decks can be evaluated by the ratio of P/P_0 according to equation (10). In this equation, φ is the average value calculated based on Equations (1) and (2). The scores of 0.8-1.0, 0.65-.08, 0.5-0.65, and below 0.5 correspond to good, satisfactory, fair, and poor, respectively.

Calculation example. Table 1 presents a summary table of 11 Oregon Department of Transportation (ODOT) concrete bridge decks, with the current ratings by the ODOT method and the new ratings by the Corrosion and Sustainable Infrastructure Laboratory (CSIL) method. The CSIL method is explained as follows. Take bridge 09268S as an example. The tested gas permeability was 61.6×10^{-17} m^2/s. Put this value as G_{eff} in Equation (1), and set the G_c value as 6.0×10^{-17} m^2/s as a uncompromised dense concrete would feature this level of impermeability. Based on the percolation theory, the critical exponent τ is 2.0. As such, $(p - p_c)/p$ can be determined as 0.32 according to Equation (1). Put the new splitting tensile strength (1,000 psi), tested splitting tensile strength (848 psi), and critical exponent $\mu = 0.4$ in Equation (2), the value of $(\theta - \theta_c)/\theta$ will be 0.66. Then the average value of $\frac{p - p_c}{p}$ and $\frac{\theta - \theta_c}{\theta}$, 0.46, will be defined as the relative density parameter, $\varphi = \frac{\xi - \xi_c}{\xi}$. The amount of actual $MgCl_2$ deicer usage on this bridge was 105 gal/ln-mi per year and the number of F/T cycles was 25 per year. We set the α, β and γ values in Equations (11) to (13) to be 0.5, 0.2, and 0.4, respectively. If the bridges experienced coupling effects, the critical exponents should be calculated according to Equation (10). For example, if the design service life of the bridge is 100 years, and the average design number of F/T cycles is 60 times each year, the total design F / T cycles should be 100×60 = 6000 cycles. From the recent years' data, the real total experienced F / T times for this bridge were 1000 cycles over the past 40 years, then the value of f in Equation (11) is 0.5×1000/6000 = 0.08. As the percentage of the real overload trucks was 10%, the value of s is 0.2×0.1 = 0.02. For the deicer exposure, the ratio of the real quantity of the deicer usage over the designed maximum value is defined as the value of c. If the designed maximum deicer quantity is 2000 gal/ln-mi/FY (this value is determined by the designation of local standard), the value of c is determined by 0.4×105/2000 = 0.02. Therefore, the general critical exponent is 0.08 + 0.02 + 0.02 = 0.12. According to Equation (10), the average rank of the bridge is $0.46^{0.12} = 0.90$.

Table 1. Summary of the 11 concrete bridge decks by old and new ratings methods.

Bridge No.	Splitting tensile strength (psi)	Rating by ODOT	Rating by CSIL	Relative Density Parameter	Gas Permeability, (in 10^{-17} m^2/s)	Traffic (ADT)	Truck %	Year Built	Average annual no. of F/T cycles	Annual deicer usage (gal/ln-mi/FY)
09268S	848±202	5-FAIR	90% - 92% (GOOD)	0.46	61.6	56700	10	1972	25	105
00576	648±9	4-POOR	35% - 41% (POOR)	0.33	61.6	6450	8	1927	102	130
08958F	739±146	6-SATISFACTORY	87% - 88% (GOOD)	0.47	28.4	7790	11	1973	26	367
08682	606±47	4-POOR	36% - 37% (POOR)	0.37	27.3	14200	15	1962	119	2662
18940	422±31	7-GOOD	48% - 56% (FAIR)	0.20	52.6	20600	10	2002	41	2091
16440	570±55	7-GOOD	36% - 37% (POOR)	0.31	39.1	8332	33	1985	174	2058
19681	664±255	7-GOOD	53% - 57% (FAIR)	0.47	16.8	5454	42	2003	248	3006
18525	523±138	6-SATISFACTORY	78% - 81% (SATISFACTROY)	0.28	35.9	13500	3	2002	93	551
16358	589±106	7-GOOD	69% - 74% (SATISFACTROY)	0.28	73.5	12801	10	1986	26	1024
16534	465	6-SATISFACTORY	26% - 34% (POOR)	0.20	86.5	9793	16	1985	26	3784
16844	809	6-SATISFACTORY	83% - 85% (GOOD)	0.48	39.4	29440	7	1990	90	166

Microstructure analysis. The microscopic characterization of selected ODOT core samples was conducted using SEM, in order to interpret the observed engineering properties of the concrete at macroscopic level. The results shed light on the mechanisms by which the $MgCl_2$ deicer deteriorates the concrete in the field environments in the State of Oregon. Out of the 12 ODOT bridge decks, a few representative cores were chosen to illustrate the consistency and discrepancy of some concrete decks when their condition is rated by the current ODOT method or by the method in this study.

Fig. 2 and Fig. 3 show the low and high magnification SEM images of the core samples from ODOT bridges 16844 and 08682, which were evaluated as SATISFACTORY by ODOT and 90-92% by CSIL, and POOR by ODOT and 36-37% by CSIL, respectively. Fig. 4, Fig. 5, and Fig. 6 demonstrate the low and high magnification SEM images of the core sample from ODOT bridges 16440, 19681 and 16534, respectively. These three bridge decks were evaluated by ODOT as GOOD, GOOD, and SATISFACTORY and by CSIL as 36-37%, 53-57%, and 26-34%, respectively.

As demonstrated in the low magnification image (Fig. 2a), the sample cored from the ODOT Bridge 16844 featured a high density of cement hydrates (likely calcium silicate hydrates, C-S-H) and very few observable pores in the paste. In addition, as can be seen from the high magnification image (Fig. 2b), the typical lamellar shape C-S-H phase was well maintained, the surfaces of the C-S-H phase were smooth, and little other precipitates or crystals were observed. This favorable microstructure corresponded very well with the high rankings by both ODOT and CSIL for its macroscopic engineering properties.

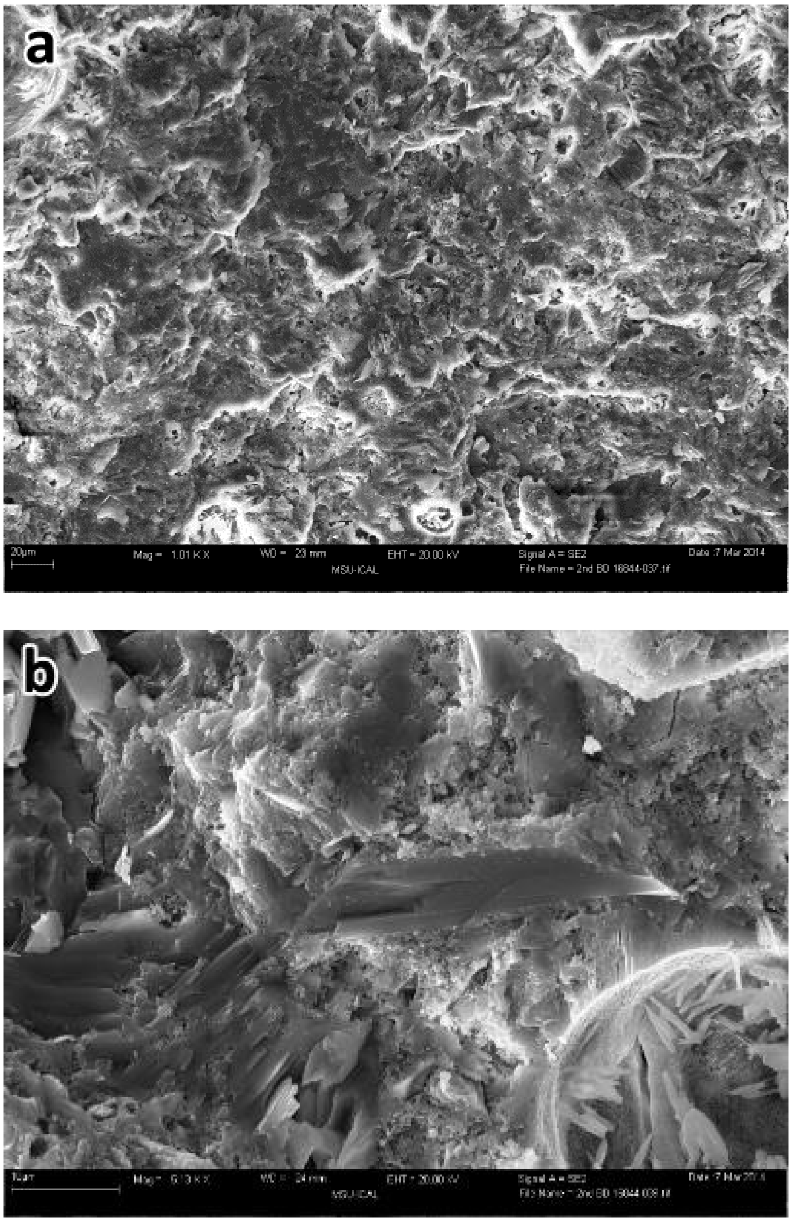

Figure 2. SEM micrographs of ODOT bridge deck 16844: a) low magnification and b) high magnification.

As demonstrated in Fig. 3, the sample cored from ODOT Bridge 08682 exhibited a drastically different microstructure than that of the core from bridge 16844. The microstructure of Bridge 08682 featured a highly porous microstructure and absence of high density cement hydrates. It can be seen from the high magnification image (Fig. 3b) that the microstructure of the C-S-H phase was no longer a dense lamellar structure but a penetrable porous structure. This deleterious microstructure corresponds very well with the low rankings by both ODOT and CSIL for its macroscopic engineering properties. This sample served as a negative reference to evaluate the physical condition of a concrete deck from the microstructure perspective.

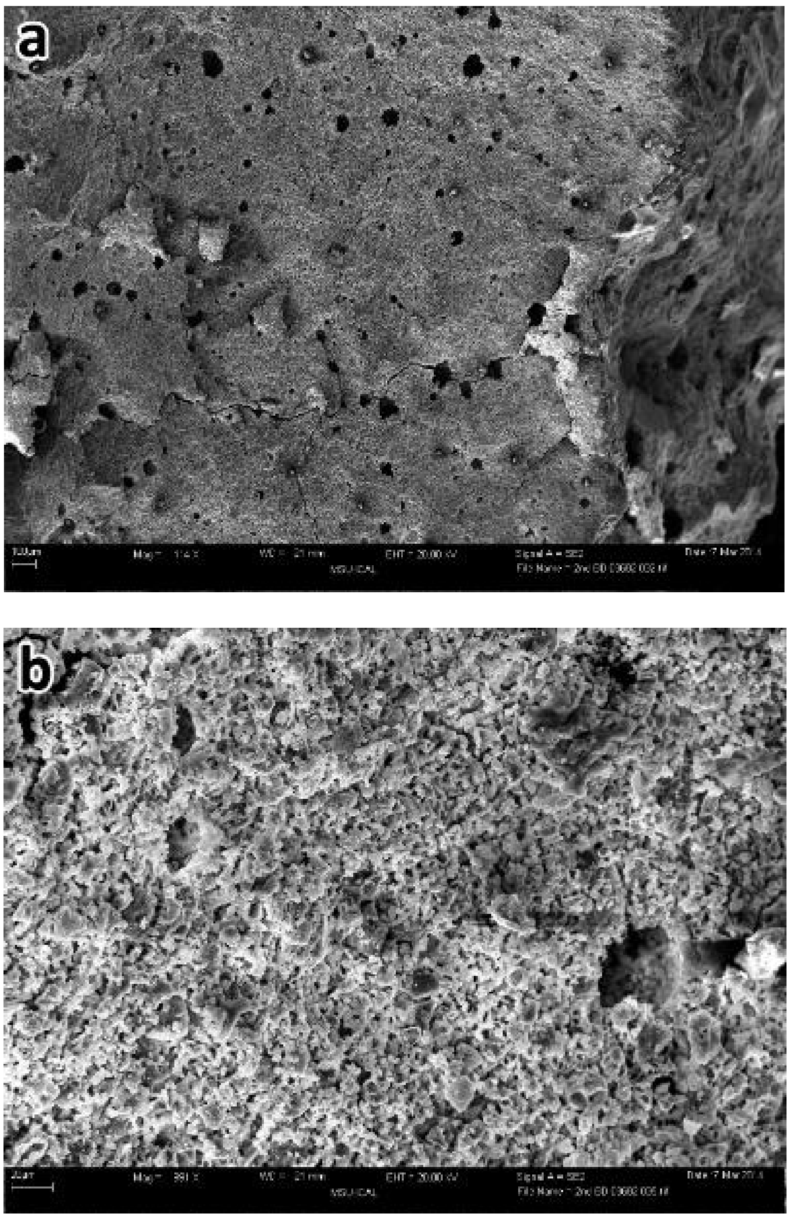

Figure 3. SEM micrograph of ODOT bridge deck 08682: a) low magnification, b) high magnification.

Fig. 4 shows the SEM micrograph of the core sample from the ODOT Bridge 16440. The microstructure of this sample exhibited typical porous structure and the presence of some precipitates on the surfaces of the C-S-H phase. The evidence suggested that the C-S-H structure had been compromised to some extent, likely by the chemical attack of $MgCl_2$. The surface of the lamellar shaped C-S-H structure was not very smooth compared with that seen in the core from the Bridge 16844 (Fig. 2b). Concrete deterioration by $MgCl_2$ solution is likely dominated by a chemical mechanism, i.e., calcium leaching induced by $MgCl_2$, which is responsible for the substantial reductions in splitting tensile strength despite of any visible surface scaling. These findings are consistent with other laboratory studies that reported the detrimental effect of $MgCl_2$ on concrete (Moukwa, 1990; Cody et al., 1994; Sutter et al., 2008) and with some limited field studies (Cody et al., 1996).

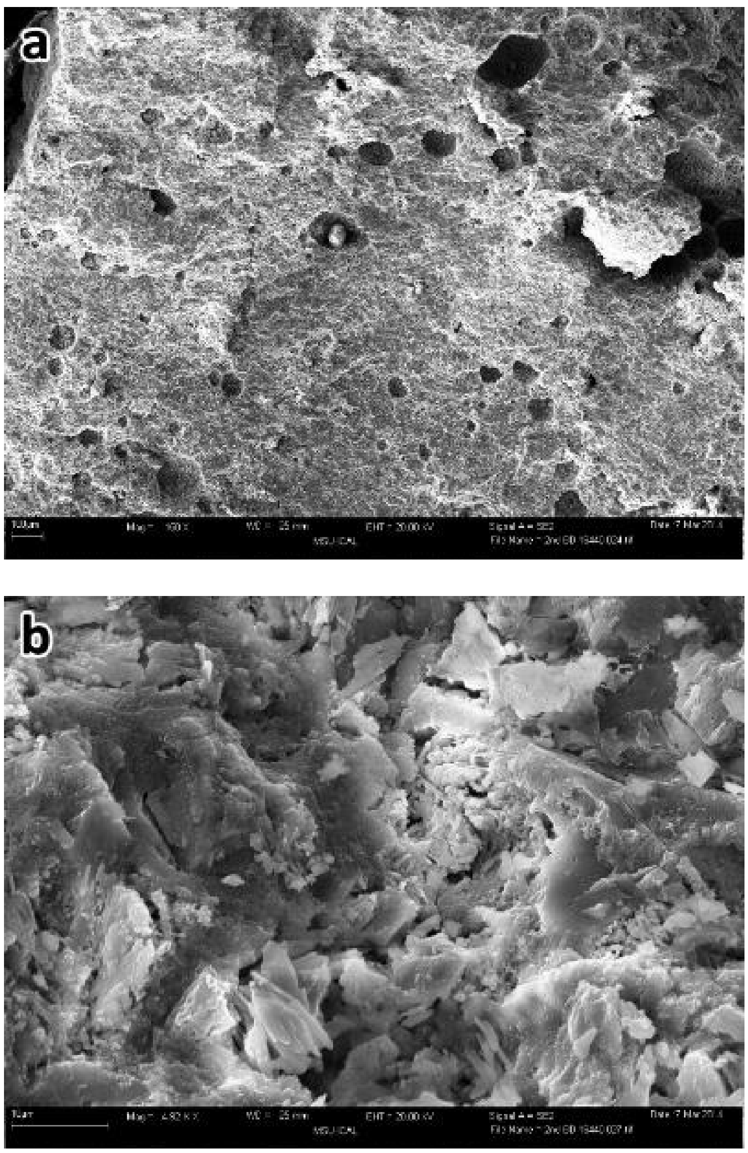

Figure 4. SEM micrographs of ODOT bridge deck 16440: a) low magnification and b) high magnification.

Fig. 5 shows the SEM micrograph of the core sample from ODOT Bridge 19681. The microstructure of this sample exhibited obvious microcracks on most of the pore surfaces but not around the pores. This implied cracks being induced by internal forces, most likely due to F/T damage in the presence of $MgCl_2$. Small amount of crystalline precipitate phases were observed (Fig. 5b). The microscopic evidence further suggested that the concrete in the field environment had been affected by both physical and chemical damages.

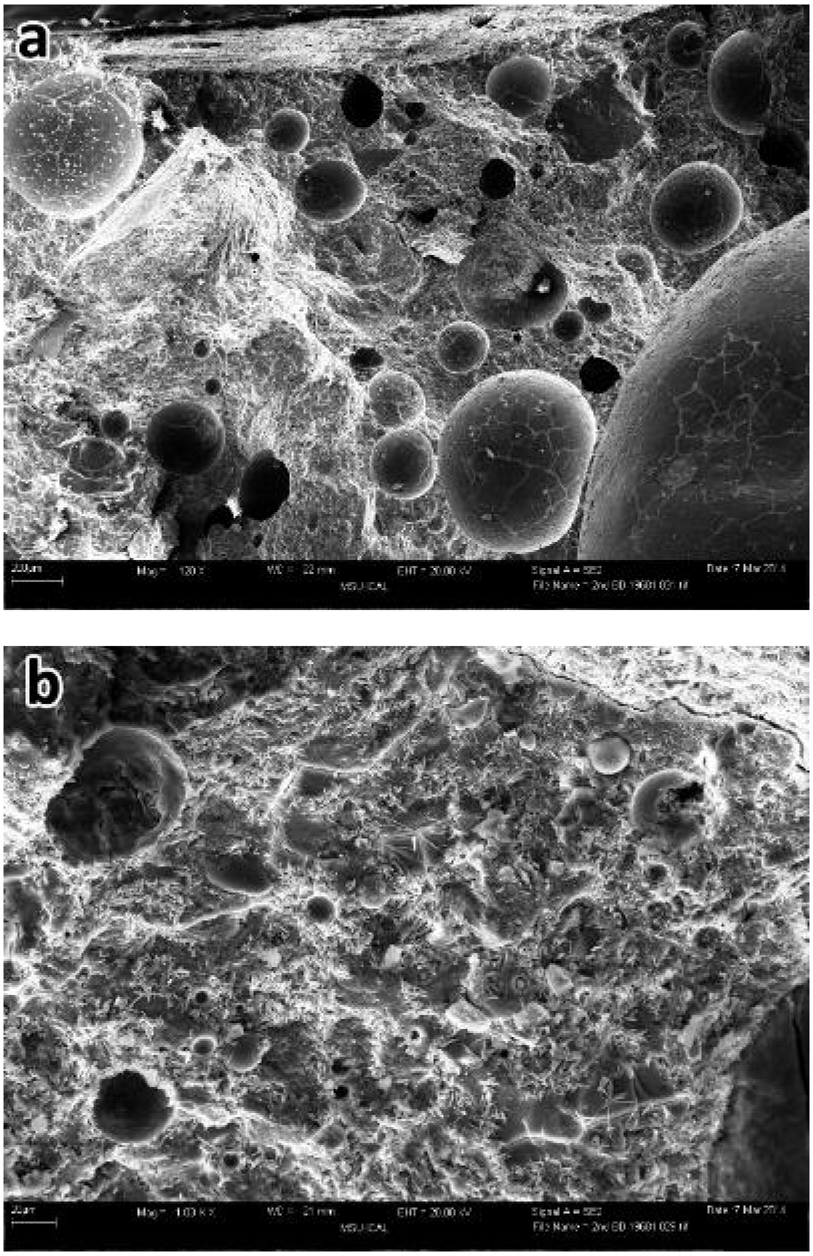

Figure 5. SEM micrographs of ODOT bridge deck 19681: a) low magnification, b) high magnification.

Fig. 6 shows obvious crystalline precipitates in the core sample from the ODOT Bridge 16534. Compared with the microstructure of 19681, there were more obvious crystalized phases that had precipitated (Fig. 6a). From the high magnification observation shown in Fig. 6b, the precipitate phases were needle-shape with a diameter of about 1 μm and a length of 10-20 μm. The precipitation process and the chemical analysis of these precipitates need to be further investigated in future studies. $MgCl_2$ may react with the cementitious calcium silicate hydrate (C-S-H gel) in the cement paste and produce a non-cementitious magnesium silicate hydrate (M-S-H). $MgCl_2$ can also react with another cement hydrate, Portlandite ($Ca(OH)_2$) and form a crystalline product known as brucite ($Mg(OH)_2$). The formation of brucite and other crystalline phases (e.g., calcium oxychloride) in confined concrete pores has been reported to induce great expansive forces and may lead to cracking of the concrete (Wakeley et al., 1992; Shi et al., 2011).

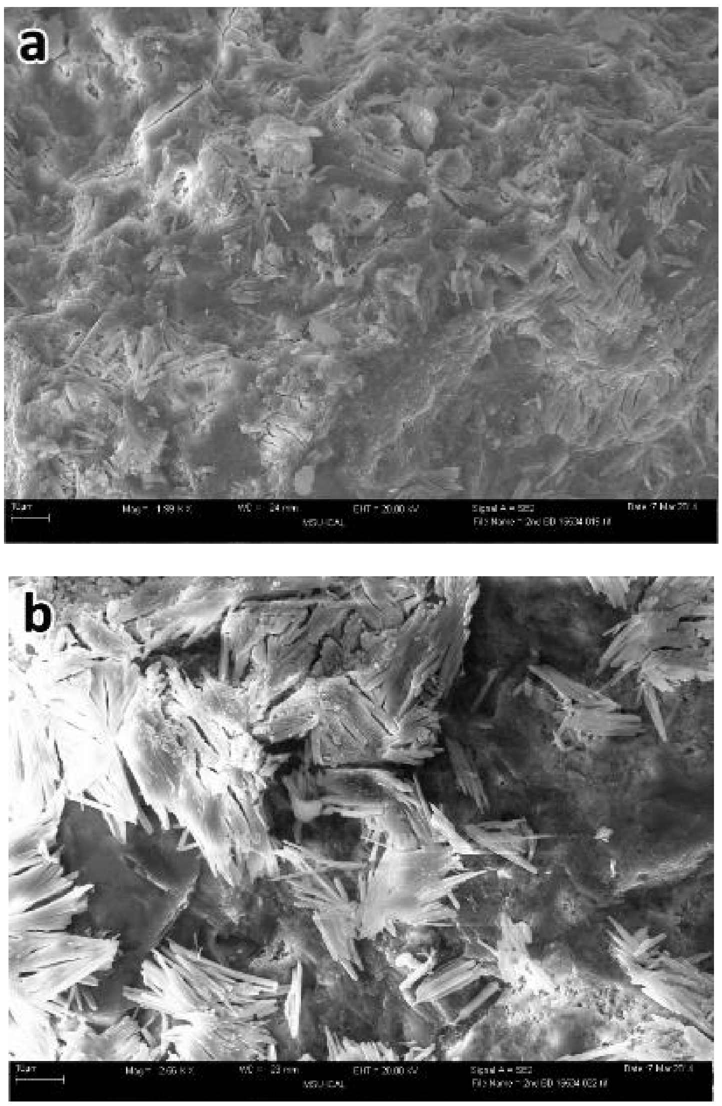

Figure 6. SEM micrographs of ODOT bridge deck 16534: a) low magnification and b) high magnification.

CONCLUSIONS

A simplistic empirical-mechanistic model was developed to evaluate the condition of bridge decks. This was made possible by combining percolation theory and the power law for strength and permeability to accommodate input parameters of the number of F/T cycles, ADT, deicer usage, and engineering properties measured from core samples (e.g., splitting tensile strength and gas permeability coefficient).

This work demonstrated that the current methods used by ODOT for ranking the conditions of concrete bridge decks may not be suitable for some decks which were exposed to the combined effect of F/T cycles and $MgCl_2$ deicer. We identified some decks that had an ODOT ranking of *GOOD* or *SATISFACTORY* but ranked as low by using our prediction model (e.g., below 50%). The mechanical testing and microscopic characterization confirmed the validity of the new model, as it better captures the concrete's internal damage that shows little signs of surface distress.

For future improvements of this model, it is desirable to adopt a probabilistic approach to predicting the chloride contamination of concrete, concrete deterioration due to deicer, and chloride-induced rebar corrosion (Stewart and Val 2003). Such a reliability-based approach takes into account the uncertainties associated with various fundamental stochastic processes and interactions, and ensures that the probability of concrete failure is kept at an acceptable level. According to Attwood et al. (1991), "reliability, defined as the probability of survival in a given period of time, forms the basis of most design codes". Furthermore, fatigue damage models based on the continuum damage mechanics (Li et al. 2001) and/or transport-based concrete durability models may be modified and adopted for predicting deicer-induced damage to the concrete matrix.

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Environmental and Energy Concerns for a Life Cycle Analysis of Transportation Systems

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Abstract

The engine of any economy is good transportation infrastructure and the supply of energy to support the freight transport and traveling public. Transportation infrastructure network assets have been the backbone of passenger mobility, vital for freight transport, and an important contributor to the nation's economy. Transportation infrastructure systems in a "sustainable" world require engineers to minimize demand on natural resources, energy consumption, and adverse impacts on the environment. This paper presents key results of a case study of technical feasibility and economic viability of an alternative transportation corridor on the Gulf Coast of Mississippi. The results of the life cycle costs and benefits favor a commuter rail service on a shared rail corridor. Added benefits include reduction of auto traffic and auto emissions. Additionally, this paper discusses the feasibility of utilizing piezoelectric materials in the harvesting of lost energy from the vibrations produced by train-rail corridors. The use of the piezoelectric sensor is a well-established technology for counting traffic on roads. In recent years, piezoelectric material has emerged as one of the most promising methods for energy harvesting from traffic-induced vibrations. This paper presents a second case study that investigates the enhancement of the piezoelectric material by coating it with a special nano particle mixture. The results indicate a more than 400% improvement in energy harvesting compared to the traditional piezoelectric material. This provides a great opportunity to tap into this source of energy being lost to the environment in the form of vibrations from train-rail corridor systems.

INTRODUCTION

Highway and rail networks have been the backbone of passenger mobility and freight transport, and have provided economic competitiveness of the United States in the global marketplace. The competitive edge linked to efficient transportation is fading out as American enterprises are trying to cope with transportation system capacity limits and tough competition from abroad. Key factors of current transportation congestion and emission concerns include migration of rural population to cities and urban areas, reliance on personal auto, and extensive use of long-haul trucks to transport manufactured products and distribute commodities for supply chain stakeholders. Major mobility problems in most urban areas include the following: high traffic congestion on roads and highways during peak hours, increased volume of commercial traffic and safety risks to other auto motor traffic, high level of vehicle emissions, and lack of public transportation options for the underserved. Coastal communities, such as the Mississippi Gulf Coast cities, face traffic congestion during hurricane evacuations, which is a major hurdle for federal, state and local emergency management agencies (MDOT 2011). The carbon emissions from transportation account for 27% of all anthropogenic GHG emissions in the USA. Transportation infrastructure systems in a "sustainable" world require engineers to minimize demand on natural resources, energy consumption, and adverse impacts on the environment. Linear transportation corridors provide great opportunities to produce renewable energy that can be used to substitute fossil-fuel based energy for electric power demand in the corridor (Karunarathna et al. 2013). This paper discusses the life cycle assessment of environmentally sustainable transportation systems assets and the potential of energy production.

OBJECTIVES AND SCOPE

The primary objectives of this paper are to: (1) address environmental sustainability dimensions of multimodal infrastructure assets, (2) enhance commuter passenger mobility by providing rail service and reducing road traffic congestion, and (3) investigate harnessing energy from vibrations due to trains moving on the rail corridor. The study specifically examines economic viability of sustainability of passenger and commuter rail revival for the Mississippi Gulf Coast cities. The rail study and the study of recovering energy from transportation corridors are both being conducted in two projects funded by the National Center for Intermodal Transportation for Economic Competitiveness (NCITEC) and U.S. Department of Transportation (http://www.olemiss.edu/projects/cait/ncitec/).

CASE STUDY OF PASSENGER RAIL REVIVAL

Background. Travel time delay due to congestion is a major problem in urban areas, which increases undesirable emissions and causes stress on auto commuters. The high commercial traffic volume increases general congestion on highways and safety risks to other auto commuter traffic. Congestion is an indicator of the deficit between travel demand and capacity, which is measured in terms of traffic density or travel time. It is estimated that commuters spend 46 hours annually stuck in traffic congestion, and waste 5.7 billion gallons of gas annually. This increment in congestion directly affects the increase in air pollution and carbon dioxide (CO_2) emissions, mobility costs, user operating costs, public health costs, and other societal costs (Uddin 2012). The following 2010 key statistics related to travel and mobility in the United States are:

- Trillion dollars on freight logistics (10% of U.S. GDP)
- 65% of goods originate or terminate in cities
- Congested Travel (% of peak VMT) 59

- Congested System (% of lane-miles) 47
- Total Delay (1000s of person-hours) 41,808
- Delay per Peak Auto Commuter (persons-hrs) 39
- Annual Excess Fuel Consumed (1000 gallons) 33,804

The transportation sector accounts for 28% of all anthropogenic greenhouse gas (GHG) emissions in the U.S. On-road vehicles contributed 84.5% of all transportation related GHG emissions in 2008. It is estimated that 75% of global GHG is produced in cities (Uddin 2012). The mobile sources, industrial sources, and urban area sources of air pollution produce adverse impacts on air quality, public health, and other detrimental consequences of global warming. The costs of related medical and public health care, lost productivity, environmental cleanup, and other societal costs are enormous (Uddin 2006).

About 80% of the current population in industrialized countries lives in densely populated cities and metropolitan areas because cities provide tremendous opportunities. Higher density cities relieve pressure on surrounding natural habitats and biodiversity if sustainability goals are given priority in urban planning and infrastructure development. Uddin (2013) shows that per capita vehicle emissions in rural cities in the southern states are significantly higher than that calculated for large cities with higher population density and good mass transit operations, such as New York and Chicago (Figure 1). The problems inherent to automobile transportation stem from the use of petroleum derived fuel (gasoline and diesel) for power. The burning of petroleum-based fuel emits primarily three pollutants: hydrocarbons or volatile organic compounds, carbon monoxide, and nitrogen oxides. These products contribute greatly to pollute ground-level air and cause smog, ozone, cancer, lung disease, and respiratory disease. Also, they produce greenhouse effects. These greenhouse gases are well known contributors to global warming, stratospheric ozone layer depletion, increasing sea levels and temperatures, and climate change mechanisms. The CO2 levels are used to measure the sustainability performance of transportation corridors Uddin (2013).

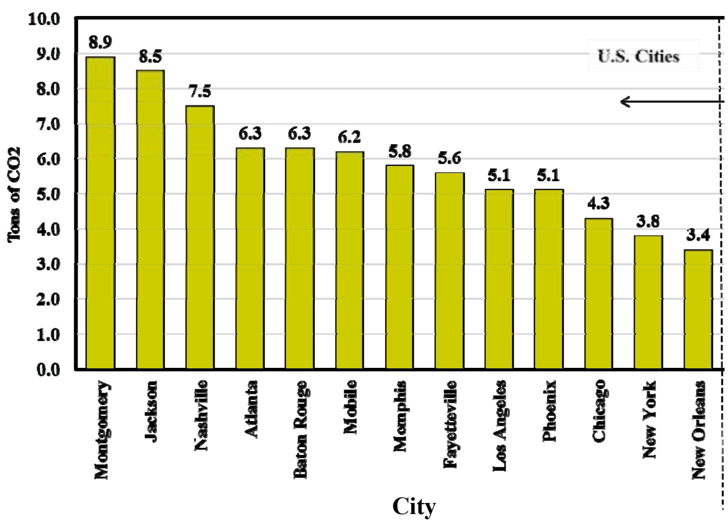


Figure 1. Annual per capita roadway CO_2 emissions, 2007 (Tons of CO_2).

Needs for sustainable passenger mobility infrastructure on gulf coast. The Mississippi DOT's strategic multimodal planning report (MDOT 2011) indicated major traffic congestion problems in and around Mississippi Gulf Coast cities. These include: high congestion on highways during peak hours, traffic bottleneck for staff and businesses serving major employers like Gulfport Port, increased volume of commercial traffic and safety risks to other auto motor traffic, high level of vehicle emissions, and lack of public transportation options for the underserved. We need to find ways to integrate passenger/ commuter rail with the auto traffic which can ease auto travel demand on the existing road corridors, offers economically competitive and safer travel, and reduces air pollution. Safety is a concern on the Gulf Coast too, especially I-10 and US-90 which are used by all casino patrons. About 14.8 million patrons visited in 2012. According to the 2012 Census data for Mississippi (ATS 2012), about 82% auto traffic is reported in the Gulf Coast counties and 11-13% person-trips by car/van pool. The combined results for all three counties for estimate vehicle trips follow:

- Daily total person trips: 390,370
- Daily total vehicle trips: 364,993
- Daily total Vehicle trips by Rail: 34,185 (estimate)

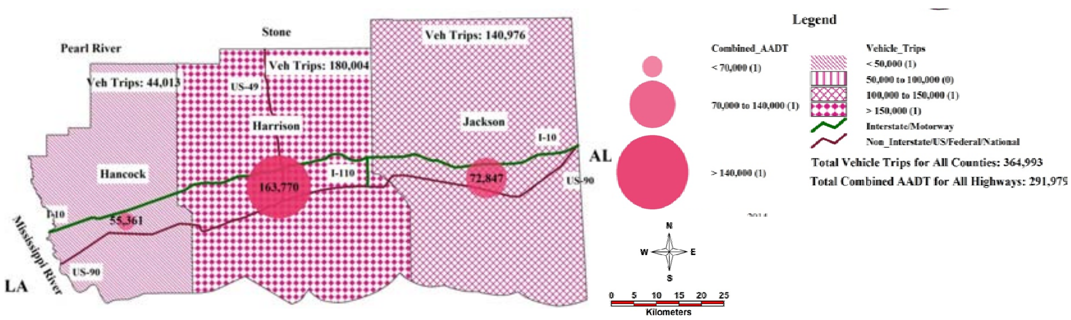


Figure 2. Spatial map of average AADT of major highways and vehicle trips in all three Mississippi Gulf Coast Counties, 2012.

Figure 2 shows the results of an extensive analysis of Census travel and community survey data based on person-trips in each county, travel mode patterns, and traffic volume demand on I-10 and US-90 highways. Typically over 49,000 vehicles travel daily on I-10 just east of the Mississippi state line and similarly over 46,000 traffic vehicle counts are reported at the western state line (Uddin 2014). The existing Coastal Transit Authority (CTA) operates its fleet of 29 buses mostly in Harrison County and it cannot serve the needs of most of the commuters in the three coastal counties. The Mississippi DOT's strategic planning reports indicate that most of the Interstate-10 corridor through these counties has average speeds (in both directions) at or below 55 mph (88 kmph) that is 21% lower than the posted speed of 70 mph (112 kmph). This indicates significant traffic congestion during rush hours. This leads to fuel wastage and increased travel time by auto road users, as well as public health hazards associated with vehicle emissions. The ultimate long-term solution for these major problems of the Mississippi Gulf Coast is to implement an efficient and sustainable alternate passenger transportation system that can also reduce travel demand on the highways.

Evaluation of public transport options. A detailed review was undertaken for the existing rail and bus transit technologies being used in several U.S. cities. The study revealed that trams and street rail cars, bus rapid transit (BRT), and high-cost light rail transit (LRT) are not feasible for the rural area on the Mississippi Gulf Coast due to degradation of highway traffic capacity from lane occupancy and rail track intrusion on shared pavements, as well as added safety risks of mixed multimodal traffic with frequent speed zones near and in populated areas. The next option was to investigate the economic viability of the discontinued passenger rail service. Currently, the Amtrak sunset service from Miami to Los Angeles through New Orleans is not operational. It was first interrupted in 1993 after the worst rail disaster in Amtrak history near Mobile, Alabama. It served the Gulf Coast triweekly before it was suspended during the 2005 Hurricane Katrina disaster. It did not serve regular commuters on the Gulf Coast similar to the daily commuter rail services operated between other large cities on the East Coast or Bay area in northern California. Passenger rail service produces relatively lower CO_2 emissions than the emissions from the road traffic, as well as it reduces congestion on roads and auto fuel consumption (Uddin 2012). Next, technical feasibility and economic competiveness evaluations are presented for passenger service on the Mississippi Gulf Coast using life cycle assessment (Uddin 2014).

Life cycle analysis for value engineering studies of transportation corridors. Life cycle analysis (LCA) or "whole life" approach, used for the first time in the 1997 pioneering book Infrastructure Management System, has been expanded to include life cycle sustainability performance of transportation systems (Uddin et al. 2013). This approach integrates design, construction, renovation and life cycle maintenance phases of physical transportation infrastructure assets serving the society. The life cycle costs and benefits are analyzed by discounting all costs or benefits to a present worth value using the following equation (Eq. 1). The present worth of the life cycle value of a project for number of years (n), is total discounted initial capital cost C, a yearly maintenance cost M, a future rehabilitation cost F, and a salvage value S. Where, PWFA is present worth factor for annual series of equal expenditure (or benefit) and PWFS is present worth factor for a single expenditure (or benefit) in a future year.

$$P = C + M\,(PWFA) + F\,(PWFS) - S\,(PWFS) \tag{1}$$

$$PWFA = \left[\frac{(1+i)^n - 1}{i(1+i)^n}\right] \qquad PWFS = \left[\frac{1}{(1+i)^n}\right]$$

The above life cycle analysis approach is used as a part of Value Engineering (VE) to assess life cycle cost and benefit of an environmentally sustainable passenger transport system on the Mississippi Gulf Coast. The VE methodology is used to evaluate alternative technologies and methods for achieving reduction in overall life cycle costs without compromising safety and functional performance (Uddin et al.

2013). The researchers of the Center for Advanced Infrastructure Technology (CAIT) applied the VE approach to study economic viability of sustainable transportation systems.

This paper presents technical and life cycle economic competiveness evaluations for alternative strategies of passenger rail/commuter intercity rail service in East-West (E-W) Coastal corridor south of I-10 and North-South (N-S) Corridor east of US-49. These rail corridors already exist and are used by freight train lines. The commuter rail service will be implementable through shared used agreements with the freight rail operators. Data on capital costs, maintenance frequency, and annual costs of maintenance and operations are derived from national transportation databases. Sales tax revenue, casino patrons, and gaming revenue data were collected from Mississippi state resources (Uddin 2014).

Table 1 shows salient features of the value engineering analysis of E-W rail line using Eq.1 for present worth (PW) life cycle (a) costs and (b) benefits. These amounts are calculated using a spreadsheet for an analysis period of 10 years at an annual discount rate of 5%. More details of LCA for the analysis periods of 5, 10, and 20 years are presented by Uddin (2014). The analysis does not include the boost to the state economy by future attracted casino/tourist traffic and potential new manufacturing jobs. Indirect factors are not evaluated in this study, such as emission related health costs, water contamination due to surface runoff, and other social justice problems. However, most of these costs will be reduced compared to the base scenario of road and other mass transit alternatives (transit, BRT and LRT) which share the road. Figure 3 shows the two rail lines serving employers and passengers in the region and casinos on the coast.

Table 1. Present Worth Cost and Benefit Analysis for E-W Commuter Rail Alternative for 10-Year Analysis Period.

(a) Life Cycle Cost Items		Commuter Rail E-W (200 km)		
		$ Million	PW Factor	PW $ Million
1. Infrastructure Capital Cost (no water crossing)	Initial	335.2	1.0	335.2
2. Annual Operating Cost	per year	6.4	PWFA	49.4
3. Annual Maintenance Cost	per year	2.6	PWFA	20.2
4. Major Overhaul	every 5 years	1.2	PWFS	1.7
5. Train/Veh Replacement (After 75 years)	Note: Salvage Not Considered		Total Cost: 406.5	

(b) Life Cycle Benefits (20,000 Riders/Day)	**Commuter Rail E-W** (Assuming electrified rail)		
	$ Million per Year	PW Factor	PW $ Million
1. Revenue, Direct	74.6	PWFA	576
2. Revenue, Gaming	152.5	PWFA	1,177
3. Revenue, Sales	6.6	PWFA	51
4. Savings From Fuel	67.9	PWFA	524
5. Economic Benefit	42.4	PWFA	327
Note: Economic Benefit include added jobs; it does not include visitors/casino patrons added.		Total Benefit:	2,656

The breakeven point of the commuter rail service is expected within 10 years of operation considering direct revenues only (item 1) in Table 1 (b). The direct revenues include fare, advertising on train and stations, annual fee charges to concessions on stations and shuttle service operators). Both rail lines will become financially self-sustaining within 10 years. In subsequent years these rail lines will generate annual revenue for the state transportation agency, unlike non-revenue management of public road infrastructure. The final recommended passenger rail strategies include:

- a commuter rail “Casino Train” in E-W corridor of the existing CSX rail from New Orleans to Mobile and
- a second commuter rail line “Beach Train” in N-S corridor of the existing KCS rail from Hattiesburg to Gulfport in Mississippi.

The two proposed rail services are expected to operate at 60 mph (96 kmph) under agreements with the existing freight rail lines and preferably with rail electrification system. The commuter rail service will have daily operations at each hour in morning and evening peak hours and every two hours in between and until midnight. These recommended rail strategies are intended to reduce auto traffic by 34,000 vehicles daily on major highways serving Mississippi Gulf Coast cities. An average commuter makes the decision of modal choice by comparing the out-of-pocket costs and travel time to destination. Therefore for any mass transit solution to succeed, it will be important to provide good transit shuttle connectivity at rail stations and participation of employers in the vicinity. Additional benefits include providing safe driving alternatives of “Casino Train” to casino patrons on the Mississippi Gulf Coast which accounted for 52% of all gaming associated revenue in Mississippi in 2012. The recommended rail will enhance highway safety by reducing the number of autos and save lives during hurricane related mass evacuation.

Figure 3. Proposed E-W “Casino Train” and N-S “Beach Train” commuter rail lines.

Using alternative renewable energy for sustainable rail line operations. The LCA summary presented in the preceding section is based on the following sustainable operations: (1) electrified train-rail service instead of traditional diesel rail engines and (2) grid electricity substations serving train operations and as backup where other renewable energy will be used. Additionally, the following renewable energy sources will be considered to enhance sustainability dimensions: (1) solar energy panel installation on the roofs of passenger train stations to provide power for station operations and light fixtures in and around the premise and (2) piezoelectric sensor installation within the rail track to harness electric energy to provide electrical power to rail signals, light fixtures, video surveillance systems, and Wi-Fi communication hubs along the rail corridor. The following case study presents the on-going research in the NCITEC project for the enhancement of piezoelectric electricity generation technology.

CASE STUDY OF ENEGY RECOVERY FROM RAIL TRANSPORT ASSETS

Background and need. Enormous energy is wasted in the form of vibrations from trains running on rail road tracks. Recovering this energy from the lost vibration could have considerable economic impact. This recovery process will also be economical, efficient, safe, and sustainable. Since the 1990s the piezoelectric material has been used in all states and some Canadian provinces. It has been embedded beneath the pavement surface of more than 1,000 in-service road sections to measure traffic counts and axle classifications as a part of the Long Term Pavement Performance (LTPP) research conducted by the National Academies and supported by the U.S. Department of Transportation (ILDOT 2004). Therefore, its use in transportation corridors poses no threat with respect to existing environmental regulations.

The use of traditional piezoelectric materials to capture this loss of vibrational energy has shown some promise because of their unique ability to produce electricity directly by deformation caused by vibrations and moving loads. Currently, the piezoelectric material is the most popular energy harvesting material employed in the field and to date there is no legislation restricting its use. Several research studies (Glynne-Jones et al. 2001; Marinkovich and Koser 2009; Roundy and Wright 2004; Sodano et al. 2002; Zheng and Xu 2008) have found that piezo electric materials can generate power in the range of 40 to 630 microwatts/cm^3 when subjected to low frequency vibration. However, improvements are being explored in the current NCITEC research project.

The research group in the Mechanical Engineering Department at the University of Mississippi (UM) has developed a special coating for energy harvesting improvement (Bhagmar 2009; Devagowda 2007; Karunarathna et al. 2013; Sharma 2012; Singh 2009) by using nano particles in a ferrofluid based mixture. The objective of this case study is to demonstrate that the ability of traditional piezoelectric materials to generate power can be enhanced by coating them with this special nano particle mixture.

Methodology. Lead zirconium titanate (PZT) is the piezoelectric material that has been selected for use in this study. A schematic view of the makeup of the PZT composite is shown in Figures 4 (a) and (b).

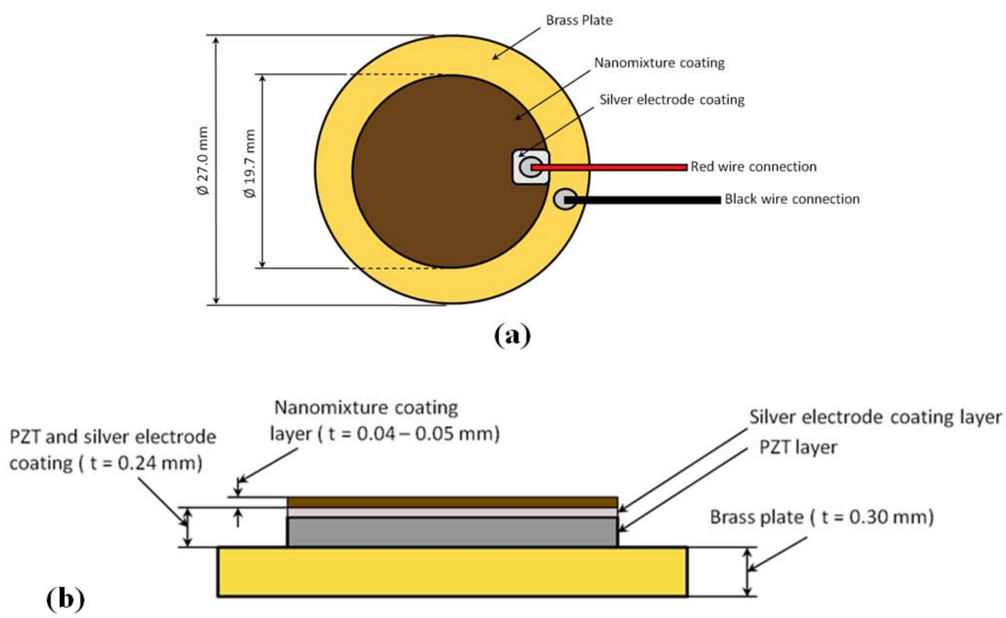
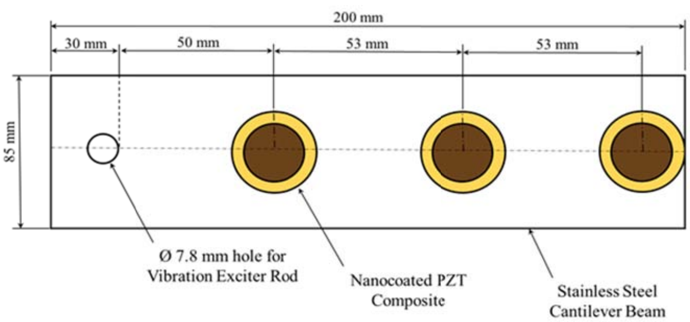


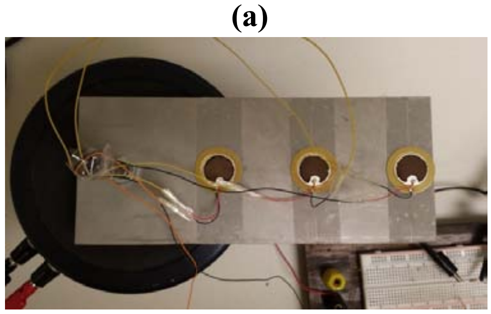
Figure. 4 (a) Schematic of PZT composite (top view); (b) Schematic of nano coated PZT composite with thickness t of the various layers (side view).

The optimum composition of the nano coating mixture in regards to power generation was earlier determined to be 60 percent ferrofluid and 40 percent ZnO by researchers at the University of Mississippi (Sharma 2012). This optimum composition has been used in the present study. The nano coating mixture was applied onto the PZT composite substrate and then cured under ambient conditions for 36 hours. Three PZT composites were next placed on a rectangular stainless steel cantilever plate having dimensions of 85 millimeters width, 200 millimeters length and 0.50 millimeters thickness. A schematic of the PZT composite cantilever system and its actual picture are shown in Figures 5 (a) and (b) respectively.

The experimental laboratory setup for collecting the energy output from a three PZT cantilever system is shown in Figure 6. It was used to collect energy from the vibration of a coated and an uncoated PZT cantilever plate in the range of 20 Hz to 1000 Hz.



(a)



(b)

Figure 5. (a) Schematic of rectangular stainless steel cantilever plate with three nano coated PZT composites; (b) Actual picture of rectangular stainless steel cantilever plate with three nano coated PZT composites.

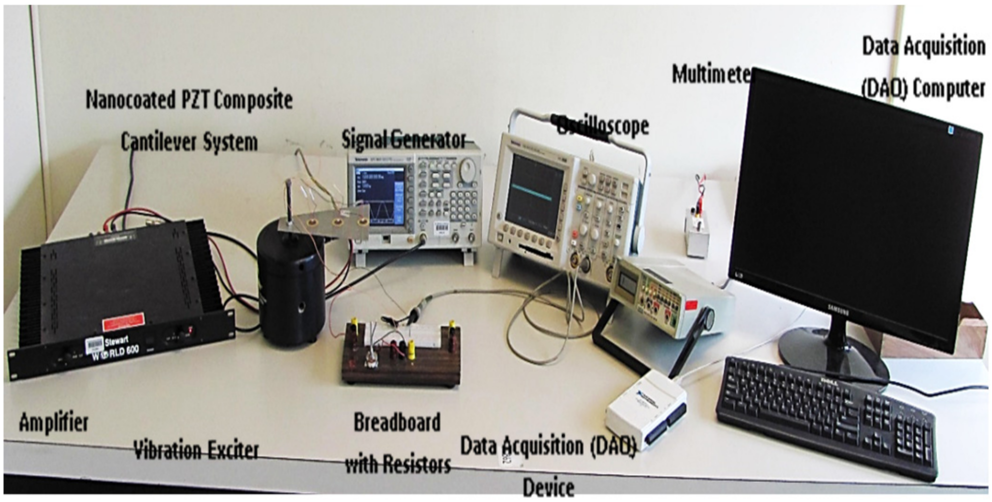


Figure 6.Experimental laboratory setup.

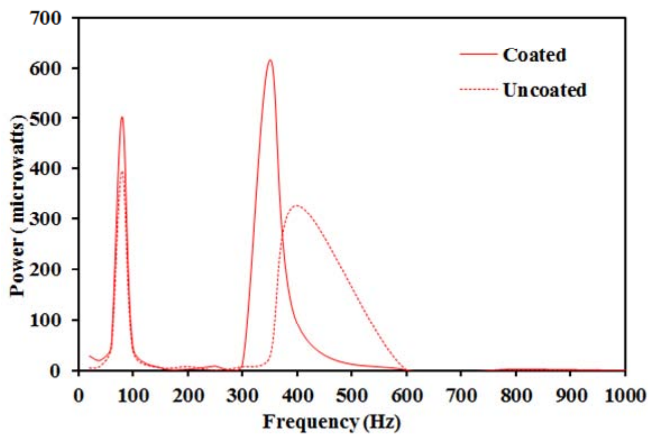


Figure 7. Power generation profiles for uncoated PZT rectangular cantilever system and nano coated PZT rectangular cantilever system at different frequencies.

Results. The experimental results for the nano coated and uncoated PZT composites are shown in Figure 7. The two highest power output values for the uncoated system were 396 microwatts at 80 Hz and 327 microwatts at 400 Hz. For the coated piezo system, the highest values were 503 microwatts at 80 Hz and 616 microwatts at 350 Hz. Table 2 shows a comparison of the uncoated PZT cantilever system results of other researchers with that of the nano coated PZT cantilever system developed by the UM researchers and described in this study.

Table 2. Comparison of UM Researchers Results with the Results of Others.

Reference	Device Type	Freque-ncy (Hz)	Overall Size (mm)	Power Density ($\mu W/cm^3$)
Roundy(2004)	Resonant-Cantilever Beam	120	30x3.6x7.7	450.9
Sodano(2002)	Resonant –Cantilever	50	80x40x1.0	625

Zheng(2008)	Resonant- Cantilever Beam	150	42x22x0.8	41.4
Glynne-Jones(2001)	Resonant- Cantilever Beam	80	23x20x0.1	65.2
Marinkovich (2009)	Resonant- Tethered Mass	160-400	4x4x0.5	125
UM Researchers (current study)	Frequency-Cantilever	20-1000	12x10x5	2807

Life cycle assessment of enhanced piezoelectric sensors. Piezoelectric sensors are low-cost technology that has been used on roads for over 20 years to count and classify traffic (ILDOT 2004; Qi et al. 2013). Installed under road surface, the life cycle costs of piezoelectric sensors include: installation cost of $9,000 beneath the pavement surface and annual operating and maintenance cost of $5,000. Based on the UM research study, the current cost of nano coated piezoelectric assembly is less than $5 per unit. These implementation costs will be significantly lower for rail track applications because it will be easier to install and maintain in the middle of the track. The cost of PZT for energy harvesting will be reduced even taking into account additional cost of energy storage and transmission. For the Gulf Coast commuter rail service these costs will be offset by the reduced use of grid electricity consumption to power, rail stations, light and signal fixtures, and other Intelligent Transportation System (ITS) infrastructure including traffic video surveillance camera network and communication system. It is recommended to conduct a follow up study for the proposed rail service by examining detailed life cycle costs, energy savings, and related economic and benefits of using sustainable enhanced piezoelectric technology.

CONCLUSIONS

The economic viability study of sustainable passenger train service on the Mississippi Gulf Coast shows that up to 34,000 vehicle users within the three coastal counties can potentially use the commuter rail service at competitive fare costs and no congestion-related driving stress. The commuter rail lines can operate on profit within 10 years of full operation considering direct revenues only. They provide added attraction of a safe mobility choice to casino patrons and reduced emissions. Additionally, rail lines are most suited for safe mass evacuation of coastal communities in case of a coastal hurricane disaster.

The energy study primarily involved the laboratory comparison of the energy harvesting results from the vibration of a coated and an uncoated PZT cantilever plate in the range of 20 Hz to 1000 Hz. The laboratory results indicate that there is an overall improvement of more than 400% in energy harvesting from vibration due to the addition of the special nano coating mixture to the substrate of the traditional PZT piezo. There is a great opportunity to harvest the energy throughout the life of infrastructure from transportation-induced vibrations, such as train-rail corridor systems. It is recommended to use this renewable energy as an alternative source for providing energy for rail corridor lighting, and for powering rail infrastructure monitoring equipment and rail signal lights.

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Impacts of Specialized Hauling Vehicles on Highway Infrastructure, the Economy, and Safety: Renewed Perspective

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Abstract

In the United States, the Federal Bridge Formula (FBF) has been used to set the Gross Vehicle Weight (GVW) limit and axle weight limit for various vehicle configurations. Further, the American Association of State Highway and Transportation Officials (AASHTO) developed a set of legal loads that closely match the FBF in the short, medium, and long truck length ranges and are used to evaluate the need for bridge weight limit posting. Specialized Hauling Vehicles (SHVs) were developed in recent years and they play an important role in the trucking industry. Typically, SHVs refer to single unit (SU) vehicles with closely spaced multiple axles that can carry up to 80,000 pounds. Even though SHVs meet the requirement under FBF, these newer axle configurations were not considered in the original development of the FBF and AASHTO legal loads. As such, FBF does not adequately restrict SHVs and most likely overstress the bridges. This study was conducted to examine the current knowledge related to the impact of SHVs on highway infrastructures, economy, and safety. The key findings from the study include: (1) Relative to conventional trucks, SHVs may not induce significant damage to asphalt pavements and can potentially induce additional damage to concrete pavements; (2) Relative to conventional trucks, SHVs can induce significant damage to short span bridges particularly to short span timber and steel bridges; (3) Bridge weight limit posting is expected to increase with the allowance of SHVs; and (4) SHVs are likely to benefit economy and safety. Findings of the study will contribute to environmentally sustainable highway infrastructure in the presence of SHVs.

INTRODUCTION

In the United States, the Federal Bridge Formula (FBF, also known as Formula B) has been used to set the limit for Gross Vehicle Weight (GVW) and axle weight for various vehicle configurations. The FBF was intended to reduce the GVW of shorter trucks, thereby preventing premature deterioration of bridges and other highway infrastructures due to concentrated weight of shorter trucks. In compliance with the FBF weight limits, the maximum GVW for various vehicle configurations is determined as follows:

$$W = 500 [L*N/ (N-1) + 12N + 36]$$

Where

W= the maximum weight in pounds that can be carried on a group of two or more axles to the nearest 500 lbs.

L = the distance in feet between the outer axles of any two or more consecutive axles

N = the number of axles being considered

In addition to FBF weight limits, the basic federal weight limits also include the following components:

- GVW limit - 80,000 lbs.
- Single-axle weight limit - 20,000 lbs.
- Tandem-axle weight limit - 34,000 lbs.

Further, AASHTO developed a new concept of hypothetical trucks, called the H (with two-axles) and the HS (with three-axles) classes of trucks, aimed to facilitate the design of durable bridges. The HS20 was, until recently, the design vehicle used to design most bridges in the United States. During the course of the bridge life, some of them become aged, deteriorated or structurally deficient. In order to have a consistent assessment of load carrying capacities, all bridges are rated using a standard set of vehicles, called "legal loads". In recent years, Specialized Hauling Vehicles (SHVs) have been introduced and they play an important role in the trucking industry. Typically, SHVs refer to single unit (SU) vehicles with closely spaced multiple axles that can carry up to 80,000 lbs. Even though SHVs meet the requirement under FBF, these newer axle configurations were not considered in the original development of the FBF. Further, the "legal loads" set by the AASHTO do not take SHVs into account for bridge weight limit posting (AASHTO, 1987, AASHTO, 2004). As a result, the FBF does not adequately restrict the weight of SHVs and most likely overstress the bridges. Additionally, a recent study has criticized the current FBF for allowing too much extra weight for trucks with additional axles, even though "bridge stress is affected more by the total amount of load than by the number of axles" (Carson, 2011).

In this context, new posting weight limits, such as GVW of 54,000 lbs., 62,000 lbs., 69,500 lbs., and 77,500 lbs. have been established for SU4, SU5, SU6 and SU7 trucks, respectively, as a result of NCHRP Project 12-63 (Sivakumar, 2007). These trucks were developed to model the extreme loading effects of single unit SHVs with four or more axles (Sivakumar, 2007). Subsequently, the same study recommended a Notional Rating Load (NRL) "as a single-load model for load rating bridges for all likely Formula B truck configurations" (Figure 1). The term 'Notional' was used because it does not represent any particular truck (Sivakumar, 2007). The aforementioned SHV configurations led to revisions to the AASHTO "legal loads" for posting as depicted in the Manual for Condition Evaluation of Bridges and the

Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges (AASHTO, 2004).

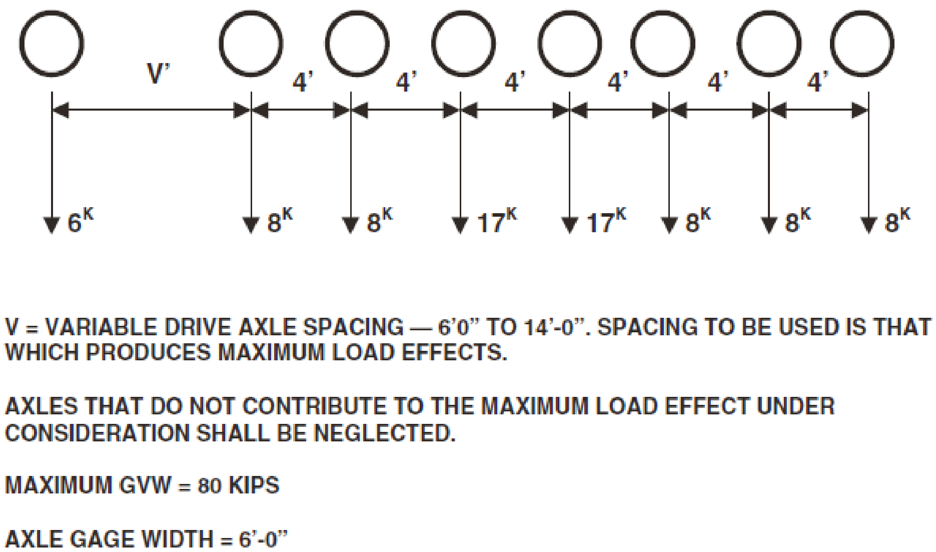


Figure 1. NRL for Single-unit SHVs that meet the FBF requirement (Sivakumar, 2007).

The development of these newer legal loads is a reactive development, which nevertheless, does not address the basic issues associated with the allowance of SHVs. The size and weight limits of heavy trucks have significant implications in terms of infrastructure costs, potential economic benefits, and motorist safety. This study aims to examine the current knowledge and practice related to SHVs with the focus on its impact on 1) highway infrastructures, 2) weight limit posting and enforcement, and 3) economy and safety. In addition to literature review, a brief survey was conducted in six identified states (Arizona, Alabama, Colorado, Kansas, Oklahoma and Texas) to document the practitioners' knowledge regarding the impact of SHVs. The findings of the study would provide systematic approaches in allowing SHVs which in turn provides environmental sustainability in maintaining sustainable highway infrastructures.

IMPACTS OF SHVS ON HIGHWAY INFRASTRUCTURES

In particular, truck characteristics such as GVW, axle weight, axle spacing, axle width and truck height play a significant role in the durability and service life of pavements and bridges. Typically, SHVs are vehicles with closely spaced multiple axles that carry heavy weight on each individual axle. In this context, it is anticipated that the SHVs' impacts on pavements and bridges are mainly dependent on their GVW, axle weight and axle spacing.

Pavements. In general, heavy trucks (including conventional trucks and SHVs) contribute to various forms of pavement damage. Of these, fatigue and permanent deformation are the important factors leading to premature deterioration of pavements, evident in the symptoms of cracking (e.g., for both flexible and rigid pavements) and rutting (mostly for flexible pavements), respectively.

Axle Weight. Axle weight is a more significant determinant in pavement damage than GVW. For example, a truck carrying 60,000 lbs. of GVW with higher single axle weight can cause more damage to pavements than a truck carrying 80,000 lbs. with less single axle weight. Further, an increase in axle weight generally causes an exponential increase in pavement damage (Gillespie, 1993, OECD, 1998, Chen and Shiah, 2001). Analysis of the variation in axle weight led to the rough generalization of fourth-power relationship. Based on the concept of ESAL many studies have been conducted to find the effect of axle load on the various pavement surfaces. A study by Gillespie (1993) found that increase in static axle weight has the greatest effect on fatigue damage. The study found that fatigue of both rigid and flexible pavements varies by a factor of more than 20: 1 over the range of axle weight from 10,000 lbs. to 22,000 lbs. This is because the fatigue damage is exponentially related to static load on an individual axle. By assuming 4^{th} power relationship, 22,000 lbs. axle load is 23 times as damaging as a 10,000 lbs. axle. The same range of static loads causes rutting to vary by a factor of 2.2: 1, because rutting is linearly related to axle load. The power of exponent has varied among different studies. For example, a study conducted by Chen and Shiah (2001) found that an eighth power law existed rather than previously determined fourth power rule for fatigue damage. Another study found that fatigue damage exponents vary between 1.58 and 2.95 with a mean value of 2 for flexible pavements for different axles. In addition, exponent of 2 was observed for fatigue cracking and exponent of 8 for rutting (OECD, 1988). It should be noted that observed exponent of 8 for rutting was done on French flexible pavements. The power of exponent also differs between different studies and pavement types.

Axle Spacing. Bringing the axles closer generally decreases the fatigue damage. In contrast, bringing axles sometimes increases the rutting effect due to the increased duration of loading period. For fatigue, when two separated loads are brought closer, the stresses induced on pavement begin to overlap. Even though the "maximum deflection of the pavement surface continues to increase, maximum tensile stress at the underside of the surface layer (considered to be the primary cause of fatigue cracking) can actually decrease as axle spacing is reduced" (FHWA, 1995). However, bringing axles closer tends to have little effect on rutting resistance due to unknown degree of increase in duration of loading (FHWA, 1995, Luskin and Walton, 2001).

For flexible pavements, distributing the same GVW over more axles (e.g., 5 instead of 4 axles) can potentially reduce the pavement damage, as this would reduce the single-axle weight. Further, axle grouping for vehicles tends to reduce their damage to flexible pavements. A study by Sebaaly and Tabatabaee (1992) found that use of multiple axles (tandem, tridem or more) reduces cracking. However, use of multiple axle increases the rutting effects relative to single axle load (Salama et al., 2006). A study conducted for the state of Michigan using the actual in-service traffic found that a vehicle configuration with 'tridem or more axle' produces more rutting effect compared with 'single and tandem axle' which produce more cracking effect (Salama et al., 2006). Another study conducted for the authority of Egyptian highways found that increasing the axle weight decreases the estimated number of load repetitions resulting in premature fatigue to roads for single axle relative to tandem axles. The study also suggests changing the configuration from single to dual tandem axle (El Sharkawy et al., 2010).

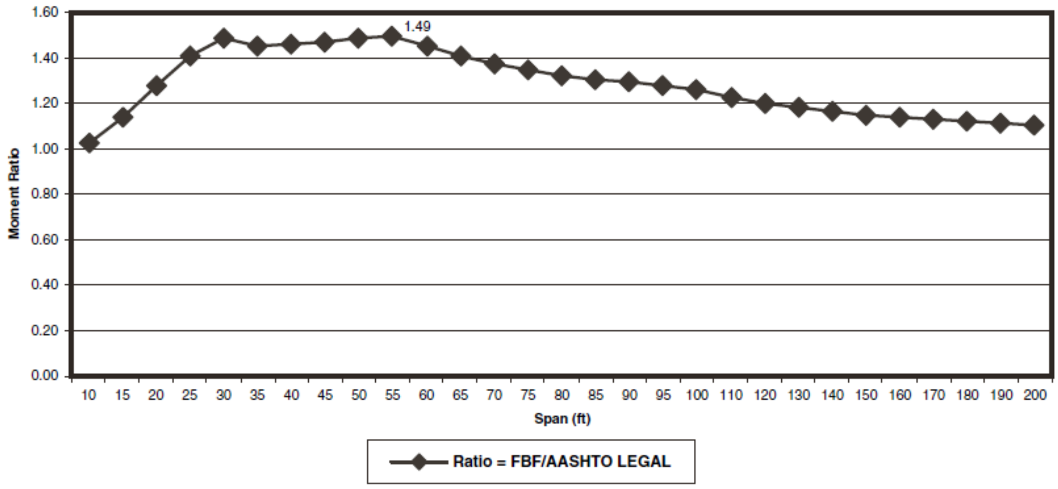
For rigid pavements, the damage is more dependent on the single axle weight than the axle spacing (OECD, 1988). However, sharing unequal load between multiple axles can induce more damage to the pavement due to generation of dynamic load (Gillespie, 1993). The dynamic loads will range from 90 to 110 percent of the static loads depending on the conditions of road (OECD, 1988). The dynamic loads can go very high, especially for vehicles without shock absorbers operating on moderately rough roads (Gillespie, 1993). In addition, effects of overlapping stress are unknown with respect to increase in time of loading period. Even though axle spacing has been known to have beneficial effects on stress reduction, pavement deterioration is complex and highly dependent on the nature of the pavement structure (USDOT, 2000).

Tire Characteristics. For each axle, the use of dual tires in place of a single tire can potentially reduce the pavement damage. Wide-space single tires are sometimes used on SHVs for easy maneuverability; but wide-spacing of tires may increase risk to pavement. Wide base single tire assemblies are on average 1.5 times more damaging than a traditional dual tire assembly with identical load axle (Smith, 1989). Sebaaly and Tabatabaee (1992)found that rutting damage between wide base and dual tire assemblies ratio varies between 1.4 and 1.6, and Bonaquist (1992) found that rutting damage ratio between a dual tire assembly and a wide base ratio varies from 1.1 to 1.5, depending on the layers of the roadway. Salgado and Kim (2002) found that super-single tires induce larger pavement strains in the pavement layers than conventional tires. Conversely, a recent study conducted on the U.S. Route 23 Test Road in Ohio concluded that tire configuration had little influence in fatigue cracking and rutting compared with the pavement structure itself (Xue and Weaver, 2013).
In addition, inappropriate use of lift axles could result in either overloading of lift axles or other axles within the vehicle, resulting in accelerated pavement damage. Lift axles are sometimes used on SHVs for easy maneuverability. For example, a study found that a raised lift axle in a tridem axle is three times more damaging to pavement than a tridem axle (deployed lift axle) (Fu and Moffatt, 2011). In addition, there are also concerns with the regulation and enforcement of SHVs with lift axles.

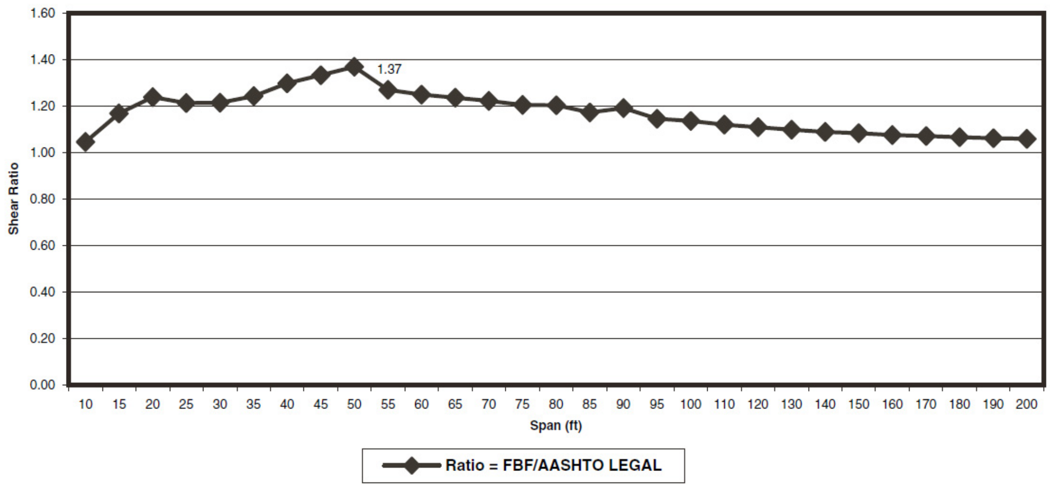
Bridges. The impact of SHVs could be significant to bridges. For instance, the recent NCHRP study (Sivakumar, 2007) used Weigh in Motion (WIM) data and survey data obtained from several states to analyze the effect of SHVs and found that these "SHVs cause force effects that exceed the stresses induced by HS20 in bridges by up to 22% and by the Type 3, 3S2, or 3-3 posting vehicles by over 50% in certain cases". Note that HS20 is the vehicle configuration that was used as the basis for bridge design in the United States up until recently and Type 3, 3S2, or 3-3 are posting loads which were and continue to be used to assess a bridge's ability to carry State legal loads. SHVs cause load effects in some bridges that exceed those of the analytical vehicles used to design and assess the bridges. In particular, the shorter bridge spans are most sensitive to the newer SHV axle configurations. Depending on the specific bridge configuration, certain types of SHVs may pose a greater risk than others and thus induce the need for bridge weight limit posting.

A study by USDOT (2000) concluded that GVW of heavy trucks plays a key role in defining the bending stress on the bridge components, in addition to the spread of axles. Laman and Ashbaugh (2000) revealed relatively weak correlation between GVW and fatigue damage potential. They also revealed that longer vehicles tend to induce an average of 15 percent of the damage induced by shorter vehicles for a given GVW, which confirms the higher risk associated with SHVs relative to conventional vehicles. Permissible loads on each individual axle can be increased by increasing the axle spacing between dual axles and triple axles, as such change would reduce the maximum moment and shear force on bridge components (Hajek and Agarwal, 1989). The study by USDOT (2000) emphasized that for both long span and short span bridges the bending stress on the bridge components is more dependent on the spread of axles than the number of axles. Laman and Ashbaugh (2000) conducted a study with 78 distinct trucks to examine the relative fatigue damage potential of steel highway bridges. The study found that weight distribution and axle spacing are the important factors in defining the damage potential for a given GVW. A vehicle with heavy and closely spaced axles will induce more fatigue damage to steel bridges than a vehicle with weight distributed over a longer length, for the same GVW. Further, closely spaced heavy axles induce more stresses on the critical points along the bridge span than the longer vehicles.

For bridges with relatively short span, SHVs can induce significantly higher moments and shears than AASHTO vehicles (Type 3, Type 3S2, and Type 3-3), as illustrated in Figure 2. This was demonstrated in a recent NCHRP study, which examined the impact of SHVs on various generic bridges with spans ranging from 10 feet to 200 feet (Sivakumar, 2007). The bridges used for analysis contained both simple and continuous spans and some with transverse members. The continuous spans consisted of two-span (two equal spans), three-span (two equal outer spans set at 0.8 times of interior span) and four-span (two equal outer spans set at 0.8 times of two interior spans). The study used 12 SHV configurations (which follows FBF limit) with outer axle spacing ranging from 14 feet to 34.5 feet with the maximum GVW limit of 80,000 lbs. SHV configurations used also included lift axles and split rear axle.



(a)



(b)

Figure 2. Moment ratios (a) and shear ratios (b) between SHVs and AASHTO posting loads (Type 3, Type 3S2, and Type 3-3) (Sivakumar, 2007).

The Alabama DOT recently conducted a study to examine the impact of a tri-axle dump truck as shown in Figure 3. Note that the "tri-axle dump truck" is a four-axle truck (19 feet outer axle spacing) with tridem axle, which can be considered as a type of SHV. In addition, the study also included SU4, SU5, SU6, SU7, NRL (with variable axle spacing of 6 ft. and 14 ft.) and HS20-44 legal loads for LFD analysis. The study employed common superstructures/standard drawing used in Alabama highways to analyze the impact of such vehicles. The study found tha some of the Reinforced Concrete Deck Girder (RCDG) structures have operational Rating Factor (RF) values less than 1.0 for "tri-axle dump truck," SU4, SU5, SU6, SU7 and NRL legal loads. A RF value less than 1.0 indicates that the bridge does not have adequate load carrying capacity for the rated legal vehicle. The impact of SHVs trucks is more evident on short span bridges. A closer inspection reveals that the RF value increases with the increase in bridge span length for both simple and continuous spans. These results are consistent with the previous findings (Sivakumar, 2007) that SHVs are more damaging to short span bridges.

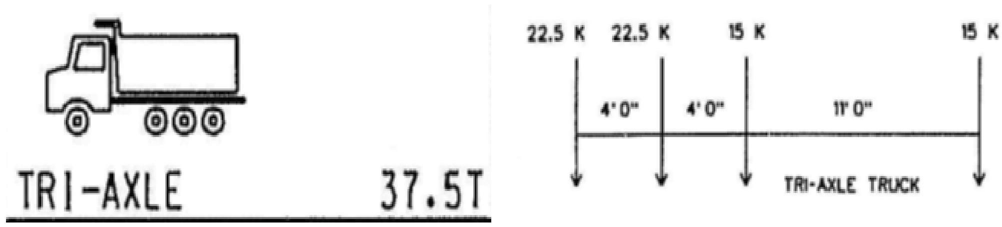


Figure 3. Tri-axle dump truck (Alabama DOT posting vehicle).

IMPACTS OF SHVS ON BRIDGE WEIGHT LIMIT POSTING AND ENFORCEMENT

Bridge weight limit posting is performed based on the outcome of load rating. It is required if the load rating indicates that the load carrying capacity of a bridge is less than "legal loads." Based on the survey results, the new AASHTO SHV models (SU4, SU5, SU6 and SU7) have not been completely adopted by the states for bridge weight limit posting. A few states are in the process of rating their bridges by including SHVs, while other states are planning to include SHVs in their future re-ratings and new bridges. A national survey conducted in 2013 for a NCHRP synthesis revealed that only nine states were using SHVs for load rating their bridges. These include: Hawaii, Louisiana, Michigan, Minnesota, Oregon, Utah, Virginia, Washington and Wisconsin (Hearn, 2014). As such, there is currently very little information available on how the introduction of SHVs has affected the number of bridge weight limit postings.

The Minnesota DOT is currently rating their bridges by including SHVs (MnDOT, 2014). MnDOT evaluated 1,310 local bridges that are most susceptible to SHVs in their first two load rating contracts. The study found that 56 percent of evaluated bridges required load posting. A large percentage of bridges that require posting are short span timber and steel beam bridges. Currently, the 3rd SHV load rating contract is ongoing, and it is about 56 percent complete (as of January 2014). Further, MnDOT proposed to begin their work on the 4th SHV load rating contract by including 890 local bridges (fiscal year 2014) and 5th SHV load rating contract by including 663 local bridges (fiscal year 2015). Figure 4 provides the percentage of bridges that require load posting for SHVs. It can be noted that the impact of SHVs is more evident on steel bridges and timber bridges. In particular, timber bridges were most affected by SHVs, and MnDOT emphasizes the growing need to remove, repair, rehabilitate or replace the timber bridges (MnDOT, 2014).

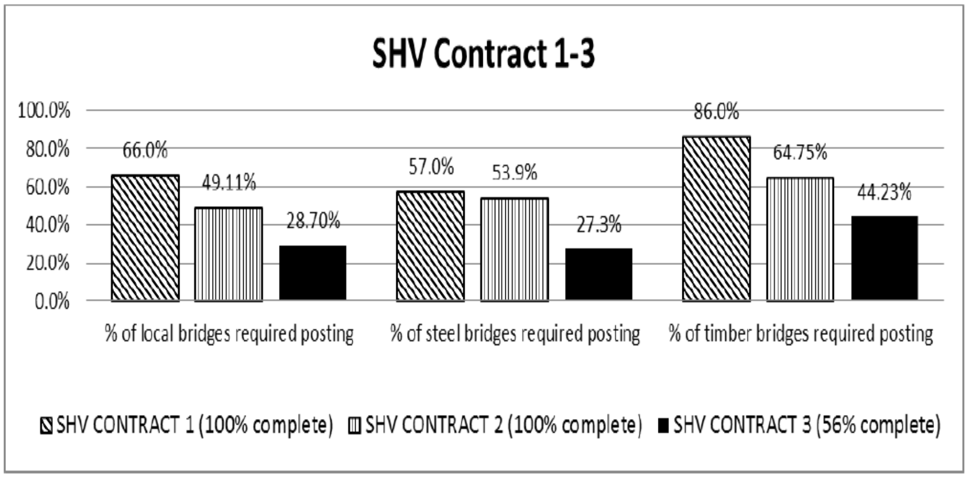


Figure 4. Results of MnDOT SHV load rating of local bridges (MnDOT, 2014).

Bridge weight limit posting is expected to increase with the allowance of SHVs, especially for short span timber and steel bridges. This, in turn, is expected to have negative impacts such as more DOT resources to install and maintain signs, increased state liabilities, increased vandalism, apathy and violations. Nonetheless, there is little information in the published literature regarding this issue.

Regarding bridge weight limit enforcement pertinent to SHVs, Colorado suggests adding another sign to indicate the weight limit of SHVs. Further, the other five states do not have much information on posting signs to differentiate SHVs at this point in time. A recent report by FHWA to address questions on load rating of SHVs suggested few examples for posting signs. Per FHWA (2014), the Manual on Uniform Traffic Control Devices (MUTCD) allows states to modify posting signs that do not contain silhouettes (specifically R12-1 though R12-4) to meet SHV configurations. Below is an example provided by FHWA (2014) (Figure 5).

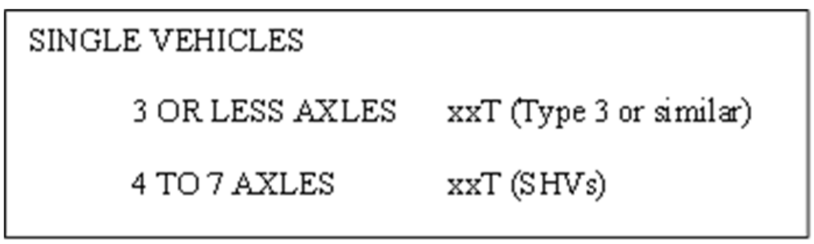


Figure 5. Example of posting sign for SHVs from FHWA.

The report (FHWA, 2014) states that "the number of axles on each line would need to be adjusted to each State's vehicle laws and appropriate level in determining the cut-offs for grouping the number of axles together". The report also suggests that states come up with their own options, such as a word sign to distinguish single unit vehicle (Type 3, SHV, etc.) and a combination vehicle (Type 3S2, 3-3, etc.). However, the posted sign should not be restrictive to other vehicles.

Another enforcement issue related to SHVs deals with lift axles in SHVs. A recent survey conducted by Fu and Moffatt (2011) found that there are no regulations for lift axles and each state has their truck regulation. Some states do not even have laws regulating their usage. In order to minimize the inappropriate use of lift axles, it is important to make truck companies accountable for violations and increasing the penalties for those who violate the lift-axle regulations.

IMPACTS OF SHVS ON ECONOMY AND SAFETY

Different dimensions of the SHV weight limits issue need to be balanced in light of local priorities and constraints. The statistics in 2004 (National Transportation Statistics, 2004) confirm that freight transport on highways greatly contributes to the economy, at more than $250 billion per year. SHVs are among the necessary freight tools to respond to specific needs of typically local or regional economies. They generally carry loads not as large or as heavy as those by longer combination vehicles, or LCVs, but offer better flexibility for freight transport. Increased truck weight limits can negatively impact the bridge and pavement network and incur additional costs to the infrastructure owners such as state DOTs, despite the potential cost savings to the truck operators (Fu, 2003).

Economy. The economic and environmental benefits can be realized by enabling truck operators to consolidate small shipments or carry heavier payloads and thus in most cases reduce operating costs (McKinnon, 2005). For example, a study was recently conducted to estimate the economic impact of a 7-axle 80,000 lbs. single unit truck (one type of SHVs) by comparing with it a base truck (5-axle, 80,000-lb semitrailer) operating on non-interstate highways (Adams et al., 2009). Adams et al. (2009) found a $2.46 million reduction in annual industry transport costs; a $0.08 million reduction in annual congestion costs; as well as environmental benefits (diverted payload of 25 million ton-miles; reduced fuel consumption of 0.04 million gallons; reduced carbon dioxide and nitrogen oxides emissions of 0.92 million lbs. and 0.96 million lbs., respectively) (Adams et al., 2009). Note that the shipper's cost savings tend to be lower for "lower weights, shorter lengths, and smaller networks" (FHWA, 1995). The economic benefits can be realized by increasing the efficiency and productivity of transportation systems (Hewitt et al., 1999, Nagl, 2007).
Numerous studies have indicated that increasing truck weight limits may significantly increase bridge infrastructure costs, but their impact on pavement infrastructure costs is less significant (Godwin et al., 1987, Luskin and Walton, 2001). Carson (2011) stated that "bridge stress is affected more by the total amount of weight than by the number of axles" . The increase in GVW would incur considerable bridge costs, with the increased need to upgrade bridges (Luskin and Walton, 2001) or the need for "accelerated maintenance, rehabilitation, or replacement work" or "posting plus enforcement" (Fu, 2003). Further, increasing the number of axles in SHVs to accommodate higher GVW (especially if they are closely spaced) is more likely to increase bridge costs. Adams et al. (2009) estimated the cost impacts due to the 7-axle 80,000 lbs., single unit truck (one type of SHVs) by comparing it with a base truck (5-axle 80, 000 lbs. semitrailer) on highway infrastructure. The study found a $2.26 million increase in annual bridge costs due to the 7 axle single unit vehicle. Walton et al. (2010) estimated a repair cost of $190 per square foot of deck area for estimating the risk of heavier trucks on highway bridges.

In contrast, increasing the truck size and weight limits "do not necessarily lead to higher pavement costs" or "can even produce savings in pavement costs" if they lead to the use of more axles. Any vehicle configuration with single-axle weight exceeding 18,000 lbs. would increase ESALs (Tang, 2012) and thus increase pavement repair costs. Luskin and Walton (2001) reported that an 80,000-lb, 5-axle tractor semitrailer "typically causes about 9 cents in pavement damage per mile of travel on rural Interstate Highways, compared with $5.90 per mile of travel on rural local roads". This illustrates the fact that the pavement cost per EASL mile is much greater for pavements designed for light duty. Bai and Center (2010) estimated an average pavement cost at approximately 2 cents per mile for heavy trucks (e.g., tractor-trailers) on a 41-mile-long highway section in the State of Kansas. Straus and Semmens (2006) estimsated an annual total of highway pavement costs between $12 million and $53 million attributable to overweight vehicles in the State of Arizona. In some cases, increasing the truck size and weight limits can even lead to decrease in pavement costs, as pavement damage is "directly related to individual axle weight rather than gross vehicle weight" (Godwin et al., 1987). For instance, the Adams et al. (2009) study found a $0.4 million reduction in annual pavement costs by comparing the use of the 7 axle single unit vehicle with a base truck (5-axle 80, 000 lbs. semitrailer).

Note that bridge and pavement maintenance, rehabilitation, or replacement activities also incur costs to highway users in terms of traffic congestion and delay (Weissmann and Harrison, 1998).

Safety. Turner and Nicholson (2009) conducted a comprehensive literature review and interviewed fifty practitioners to examine the safety implications of oversize/overweight (OS/OW) commercial vehicles. Larger and heavier vehicles were found to feature generally lower crash rates but generally more severe crashes. This trend was confirmed by the synthesis done by Carson (2011). Yet the cause-and-effect relationships between the truck size/weight and safety risks are too complicated to support any definitive conclusions. Heavier trucks tend to take longer to stop and are more difficult to operate during evasive maneuvers (OhioDOT, 2009). According to the FHWA (FHWA, 1995), higher weight limits may negatively affect vehicle stability and control and "the shorter the wheelbase lengths of the trailers in the combination, the more susceptible the vehicle is to rollover". Rollover crashes tend to be more severe and thus are a serious safety concern. Further, additional braking mechanisms can be equipped for each additional axle to help combat against the increased momentum of heavier trucks (Waldron and Yates, 2012). Adams et al. (2009) concluded that "increasing the number of axles by 20 percent (e.g., adding an axle to a five-axle combination) would reduce its crash rate by 5 percent". Further, Adams et al. (2009) compared the safety impact of various truck configurations (with increased allowable weight) on highway safety in the State of Wisconsin, using the 5-axle tractor semitrailer as a base case. The estimated safety costs are summarized in Table 1. It can be found that one type of SHV (the 7-axle 80,000 lbs. single unit truck, highlighted in bold and italics) slightly reduced the annual safety costs on highways, relative to the base case.

Table 1. Estimated safety costs for various truck configurations (Adams et al., 2009).

CONFIGURATION	ANNUAL SAFETY COSTS (million $)	
	Non-Interstate Only	Interstate and Non-Interstate
8-axle 108,000 lb. double	↓ $0.46	↓ $2.90
7-axle 97,000 lb. tractor semitrailer	↓ $0.70	↓ $4.43
7-axle 80,000 lb. single unit truck	↓ ***$0.11***	↓ ***$0.53***
6-axle 90,000 lb. tractor semitrailer	↓ $0.46	↓ $3.48
6-axle 98,000 lb. tractor semitrailer	↓ $1.52	↓ $9.40
6-axle 98,000 lb. straight truck-trailer	↓ $0.09	↓ $0.68

CONCLUSIONS

- Relative to conventional trucks, SHVs may not induce any additional damage to flexible (asphalt) pavements and can potentially induce additional damage to rigid (concrete) pavements
- The impact of SHVs to bridge damage is mostly attributed to short span bridges (25 feet to 55 feet). In particular, short span timber and steel bridges are most vulnerable to impacts by SHVs.
- Bridge weight limit posting is expected to increase with the allowance of SHVs, especially for short span timber and steel bridges. This, in turn, is expected to have negative impacts such as more DOT resources to install and maintain signs, increased state liabilities, increased vandalism, apathy and violations.
- The practice of posting signs to differentiate SHVs is not well understood at this point of time, because there are few examples to consider.
- Increasing SHVs weight limits is most likely to slightly benefit the economy, due to increased efficiency and reduced transport costs. SHVs are likely to increase bridge costs as a result of accelerated need for maintenance, rehabilitation, or replacement of bridge components, but they are unlikely to increase pavement costs unless single-axle loads exceed 18,000 lbs.
- Replacing longer trucks with SHVs is likely to benefit operator safety, as SHVs are shorter and thus easier to operate. But increasing SHVs weight limits is likely to increase their crash risk, as they would then take longer to stop and become more difficult to operate. However, assuming that fewer vehicles are run because each can have a higher payload, the reduced exposure may bring a safety benefit.

RECOMMENDATIONS AND FUTURE RESEARCH

This study aims to examine current knowledge and practice related to SHV weight limits. One of the key findings from this study is that increasing maximum weights of SHVs can significantly deteriorate bridges. In particular, local and short span bridges are most vulnerable to the impacts of increasing weight limits for SHVs. Additional studies should be conducted to enable states to balance the benefits of allowing SHV vehicles against the negative consequences to bridges and rigid pavements and against the requirements for additional enforcement and state regulatory resources on more posted bridges. For instance, more research is needed to achieve better understanding of how the introduction of SHVs would affect non-DOT agencies in terms of taxation (revenue) and operations (safety and mobility). It is noted that a complete treatment of the topic would require models that capture both the impacts (on pavements, bridge structures, etc.) and savings of SHVs.

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Review on the Toxicological Effects of Chloride-Based Deicers: Impacted Environments and Assessment Methods

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Abstract

With the growing reliance on chloride-based deicers in winter roadway maintenance operations, the concerns over their adverse impacts have been highlighted in recent years, especially the direct stress that applied chemicals bring to the receiving roadside soil, water bodies, aquatic biota, and vegetation through snowmelt runoff, infiltration and wind blow. This work started from synthesizing current knowledge on the negative/toxicological effect of chloride-based snow and ice control products to the natural ecosystem, and then put an emphasis on the examination of typical methods used to assess the toxicity of chloride-based deicers on the water quality and aquatic biota in the laboratory and field tests. The summary focused on the species to test, calculation process, and influential factors in the laboratory test, followed by the field sample collection options, timing of sampling, key variables and bio-assessments in the field test to quantify toxicological effects of chloride-based chemicals. Finally, efforts on current theoretical research and the need for further work (e.g., field monitoring and bio-assessments or bio-monitoring over time, the effective correlation between laboratory toxicity data with field data, and the relationship among key variables and resulting toxicity index) were discussed.

INTRODUCTION

Winter maintenance activities is always the first priority and necessity in cold regions to keep roadways safe and mobile for the driving public. Recent years have seen increased use of chemicals (with mainly chloride salts for freezing point depression), in the effort to improve the friction on winter pavement while reducing the direct cost of operations. With the growing reliance on chloride-based deicers in snow and ice control operations, the concerns over their adverse impacts have been more highlighted in the last decade or two. These include negative environmental impacts on the natural ecosystem (Meriano et al., 2009; Munck et al., 2010; Todd and Kaltenecker, 2012; Perera et al., 2013; Ke et al., 2013; Shi et al., 2013), corrosive effects on motor vehicles (Shi et al., 2009a; Shi and Akin, 2011), and detrimental effects on transportation infrastructure (Shi et al., 2009b, 2010a, 2010b, 2011). Of particular concern is the toxicological impacts of chloride-based deicers on the receiving roadside soil, water bodies, aquatic biota, and vegetation, as they migrate into the surrounding environment through snowmelt runoff, infiltration and wind blow.

The toxicological effects of chloride-based deicers in the field environment can be highly site-specific as they are affected by a variety of traffic, climatic, soil, hydrological, vegetation characteristics of the site. The fate and transport of the chloride salt(s) and other additives in the deicers can be very complicated, in light of the inherent uncertainties in many of the related processes and their interactions. In this context, it is necessary to provide a snapshot review of the state of the knowledge on this important subject, with a focus on impacted environments and assessment methods.

Currently many challenges remain in the laboratory and field assessments of deicer toxicity. For instance, the toxicity of a deicer product can be tested in the laboratory under well-controlled conditions, but its field toxicity can deviate considerably from the laboratory assessment due to dilution and/or interaction other chemicals in the field environment. It is very difficult to track/monitor or model the transport and fate of roadway deicers and additives. To implement a reliable toxicity testing plan, it is essential to (i) determine specific parameters of concern and develop effective monitoring strategies to understand the potential toxicity in a field scenario; and (ii) adopt suitable laboratory testing methods to approach field scenario and accurately correlate laboratory toxicity data with field data.

Now toxicity testing has not been required of manufacturers or vendors producing chloride based deicers and the associated additives (e.g., corrosion inhibitors and thickeners). Therefore, little toxicity data has been collected or published in the field of winter maintenance. Toxicity research has been conducted on similar products used in other fields including aircraft or airfield deicers (Corsi et al., 2006) and dust suppressants (Williams and Little, 2011), but available data on toxicological impacts are still very limited. The effects of the chloride chemicals on the ecosystem should be systematically assessed so as to help the winter maintenance agencies optimize their activities with less negative footprint on the natural environment. This work started from synthesizing current knowledge on the negative/toxicological effect of chloride-based snow and ice control products, and put an emphasis on the examination of typical methods used to test and assess the toxicity of chloride-based deicers on the water quality and aquatic biota in the laboratory and field tests. Finally, efforts on current theoretical research and the need for further work (e.g., the effective correlation between laboratory toxicity data with field data, and the relationship among key parameters and resulting toxicity index) were discussed.

IMPACTS TO THE ELEMENTS OF NATURAL RESOURCES

Road salts are widely used for winter maintenance operations, and after spread into the road surface, the fates of some salts will unavoidably end up being a part of the surrounding environment and bring impacts to the elements of this ecosystem.

Impact to soils. The roadside soil is susceptible to chloride pollutants via runoff penetration and airborne particles. Along with distance from the road curb, road salt concentration declines slowly with a certain pattern, e.g. exponential-like decrease (Cunningham et al, 2008; Zehetner et al., 2009). Most noticeable roadside soil contamination can be found within 5 m from road, however, the Connecticut DOT showed the airborne salt transported up to 300 ft (91 m) from roadways under heavy traffic conditions on primary or interstate highways, while in the rural and heavily forested Adirondack Mountain Region of New York, the road salt even traveled up to 172 m from the highway into wetlands possibly due to the migration of chlorides through the soil (Karraker et al, 2008).

The dissolvable and penetrable characteristics of road salt induce possible alterations in physical and chemical properties of soil through a series of chloride-soil interactions. Besides the cause of soil itself, e.g., soil type and constituent components, one of main reasons accounting for these changes is that the toxicities of road salt (e.g., Na^+ and Cl^-) could cause soil health damage and soil structure alteration. For example, when giving de-icing salt solution to soil samples taken from soil layers (0–15 cm, 15–30 cm and 30–45 cm), a trend of increasing electric conductivity values, Na^+ and Cl^- concentrations were found to be apparent, and it was proved that soil salinization would lead to unbalanced microbial community and destroyed micro-ecosystem of bulk soil surrounding plant rhizosphere (Ke et al., 2013). Another potential result of the chloride-salt interactions is the elevated heavy metal mobility in the roadside soil (for example, topsoils across a highway-forest interface northeast of Vienna, Austria), which could be achieved via a number of mechanisms including cation exchange, chloride complex formation and colloid dispersion (Zehetner et al., 2009).

With the long-time accumulation of road salt in soil, adverse consequences may also include decreased soil cation exchange capacity of urban soil (Cunningham et al., 2008), degraded soil organic matter, raised pH, and affected carbon sequestration, e.g., in the soil of temperate and boreal European forests (Clarke et al., 2009). Potential disruption of the nitrogen cycle over an extended period is another long term impact of road salting on roadside as a result of the reduction of ammonium-N retention on cation exchange sites and the disruption of the key microbial N transformation processes primarily through increasing soil pH (Green et al, 2008).

Impact to water bodies. Another detrimental effect of chloride-based deicers is their long-term risk to the receiving water bodies, due to their accumulative and non-degradable attributes. It was observed that even after the salting source was removed, the chloride concentrations in water remain high, and the slow release and the retention of chloride ions would continue even after a whole year round (Lax and Peterson, 2009). In Perera' s study (2013), the steady chloride concentration of selected case was up to 505 mg/L, which is over twice of standard Canadian drinking water quality objective of 250mg/L. While during the 2004–2005 salting season, chloride concentrations in the stream of Frenchman's Bay was even up to 2000 mg L^{-1} (Meriano et al., 2009).

It was estimated that approximate 40%-55% of salt would migrate to surface water via runoff, while the remaining would infiltrate into the hydrologic system (Meriano et al., 2009; Perera et al., 2013). Since the recharging water from snowmelt and storm runoff containing extremely high level of chloride keeps the chloride entering surface water consistently exceeds that was released, an annual average increase of chloride concentration in nearby water body was found in many high road salt application areas ranging from 0.1 to 3.0%, e.g. 1.8% of increase was observed in the thirteen lakes of Twin Cities Metropolitan Area of Minnesota (Novotny et al., 2008). As a result of this profound accumulation of chemicals, both surface water and groundwater qualities are degraded yearly. For instance, in Godwin's study, within a 47-year-period, sodium and chloride concentrations of selected cases have increased by 130% and 243% respectively (Godwin et al. 2003). In streams of Maryland, chloride concentrations was observed up to 25% of the concentration of seawater, and in New Hampshire chloride concentrations are up to 100 times greater than pollution-free streams over decades of road salt application (Kaushal et al. 2005).

Impact to aquatic biota. The inflow and deposition of road salt into roadside streams, ponds and other wetland is a continuous environmental threat for dependent aquatic biota. A series of studies have demonstrated the positive correlation between roadside stream chloride concentration and the anthropogenic source, with a high proportion to explain the variation of concentrations caused by road salt use (Todd and Kaltenecker, 2012). Elevated chloride concentration in surface water, sometimes as much as 22-fold in the course of a day, would result in severe toxicity to freshwater species, especially some environmentally sensitive aquatic species, such as freshwater mussels (Todd and Kaltenecker, 2012), amphibians (Collins and Russell, 2009), and benthic macroinvertebrates. The harmful influences for aquatic organisms may include impaired locomotor systems and reduced activities (Denoël et al., 2010), decreased survivorship and longevity (Karraker and Ruthig, 2009), unsuccessful reproduction and recovery (Gillis, 2011), and increased mortality of aquatic biota (Karraker et al., 2008). Therefore, exposure to high level chloride chemical pollutants can affect individual, population, community and even the whole ecosystem levels, which ultimately leads to a decline of community structure, species richness and aquatic biodiversity by sacrificing salt-sensitive species (Collins and Russell, 2009).

Impact to vegetation. The negative impacts of chloride-based salts on the roadside vegetation can be significant. It was reported that road salt impaired at least 15% of roadside trees in the Lake Tahoe Basin every year (Munck et al., 2010), and it showed to be the second only to the damage from insects (Eppard et al., 1992). These potential adverse effects of high concentrations of Na^+ and Cl^- could be reflected through one or some of visual symptoms, including burning of leaves and needle, foliage disfigurement, top and branch dieback, necrosis of leaves and tissue, as well as poor growth (Cekstere et al., 2010). For example, Tahkokorpi et al. (2012) found that at the end of bilberry growing season, the accumulation of Na^+ would decrease the tissue water content, anthocyanin concentrations and lipid peroxidation, and the viability of aboveground stems would decrease from 90% to 30%. It was proved that the low/mild salinity (50-100 mm) has an unfavorable influence on plant size and overall growth rate of Ambrosia artemisiifolia (Eom et al., 2013). Hanslin (2011) demonstrated that when applied during the active growth periods of vegetation, even organic snow and ice control chemicals, e.g., calcium magnesium acetate and potassium formate, can have negative effects on roadside plants as NaCl, and the direct impairment mainly goes for the establishment and growth of tree saplings. In view of the interactions of chloride-vegetation, these symptoms could be caused by the following intrinsic mechanisms: high level of chloride salts in soils can induce ions imbalance, decrease the activity of several enzymes, interfere various metabolic processes and photosynthesis, and lower chlorophyll concentration (Cekstere et al., 2008).

Relative to the high concentrations of Na^+ and Cl^- in the receiving environment, the numerous additives in the chemical formulations increased the toxicity of deicers, e.g., corrosion inhibitor 4(5)-methylbenzotriazole and other proprietary mixes of corrosion inhibitors (Corsi et al, 2006). To mitigate the resulting environmental impacts, both proactive and remedial measures should be taken to reduce potential risks and restore damaged ecological environment, e.g. conducting better urban landscape design, implementing advanced snow and ice control technologies to reduce salt usage, developing new ecologically friendly additives, installing shields for roadside trees, breeding and selecting salinity resistant type of vegetation, soil desalination and so on (Eppard et al., 1992). But on top of that, making toxicity assessment before the usage of a certain chloride-based product means of more significance for maintenance agencies and transportation officials. In this work, the efforts are concentrated on examining the toxicity test methods for water quality and aquatic biota to lay a foundation for future investigations.

TOXICITY ASSESSMENT

The most commonly used tools to assess toxicity of chloride-based products are the laboratory acute and chronic tests, which apply varying concentrations of a contaminant or mixture to a selected species. The information gained from these tests are specific to the species tested and can be used to assess damage at the cellular and subcellular level of organisms. For this reason hundreds if not thousands of species have been tested in this fashion to gather as much information as possible.

Laboratory assessment_study of species. Laboratory test species approved for short-term chronic toxicity include: fathead minnow (*Pimephales promelas*), the daphnid (*Ceriodaphnia dubia*), and green alga (*Selenastrum capricornutum*) (US EPA, 2002a). Laboratory test species approved for freshwater acute toxicity testing include: daphnid (*Ceridaphnia dubia*), daphnids (*Daphnia pulex* and *D. magna*), fathead minnow (*Pimephales promelas*), rainbow trout (*Oncorhynchus mykiss*) and brook trout (*Salvelinus fontinalis*) (US EPA, 2002b). The life stage of the test organism is important to consider, given that young organisms are often more sensitive to toxicants than adults, thus all organisms should be approximately the same age and should be taken from the same source (US EPA, 2002b)."

Calculation process. For test results to be acceptable, control survival must equal or exceed 90% (US EPA, 2002b), and for a short-term chronic toxicity test to be valid, the amount of organisms surviving and final dry weight (fathead minnow) or the quantity of young produced (daphnid), or number of cells/mL (green alga) must be met (US EPA, 2002a). The most common methods of calculating and reporting toxicity data are including:

- NOEC – No-Observed-Effect-Concentration (acute & chronic), (US EPA, 2002a);
- NOAEC – No-Observed-Adverse-Effect Concentration (acute), (US EPA, 2002b);
- LOEC – Lowest-Observed-Effect-Concentration, (US EPA, 2002a);
- EC - Effective Concentration, (USEPA, 2002a);
- LC50 (acute- and chronic-survival), (US EPA, 2002a);
- IC – Inhibition Concentration (IC25&IC50), (US EPA, 2002a).

NOECs and LOECs are determined by hypothesis testing. Common test methods include: Dunnett's Test, t-test with the Bonferroni adjustment, Steel's Many-one Rank Test, or the Wilcoxon Rank Sum Test with the Bonferroni adjustment (US EPA, 2002a). NOECs and LOECs assume either a continuous dose-response relationship or a non-continuous (threshold) model of the dose-relationship. While the LCs, ICs, and ECs are determined by point estimation techniques using: Probit Analysis, Spearman-Karber Method, Trimmed Spearman-Karber Method, Graphical Method or Linear Interpolation Method (US EPA, 2002a), LC, IC, and EC are derived from mathematical models that assume a continuous dose-response relationship. Other statistical analysis methods can be used as well.

Influential factors. Temperature of the test chamber(s) should be continuously monitored, and dissolved oxygen (DO) and pH should be monitored daily. Monitoring of salinity or conductivity, total residual chlorine, total alkalinity, total hardness, and total ammonia should also be considered (US EPA, 2002a). In addition to the type and life stage of aquatic species, the following variables are also typically considered in the laboratory investigation of product toxicity.

1) Product concentration and range

Based on the US EPA guidelines for *Whole Effluent Toxicity: Guidelines for Establishing Test Procedures for the Analysis of Pollutants Supplementary Information Document (SID),* effluent testing for toxicity should include a minimum of five effluent concentrations and a control (US EPA, 1995). Example test effluent concentrations provided include: 100% effluent, 75% effluent, 50% effluent, 25% effluent, and 12.5% effluent (US EPA, 2002a). Product concentrations to be used in laboratory testing of deicer toxicity may need to be determined using range-finding tests. Because limited research on the toxicity of chloride based deicers has been completed, range-finding tests may be necessary for each individual product. A range-finding study is an acute toxicity test of groups of five organisms that allows you to determine an appropriate concentration range for the tests to be conducted, and tests run over an 8 to 24-hr period.

2) Water temperature and flow rate

The US EPA (2002a&b) test methods specify testing temperature ranges for the chronic and acute test methods. These test methods recommend continuous monitoring of water temperature. For static non-renewal and static-renewal test methods, water flow rate is not relevant, but for flow-through testing, water flow rate can be considered and is specified in US EPA (2002a&b).

3) Water hardness

Since $MgCl_2$ and $CaCl_2$ contained in deicers can affect water hardness, there is most likely a relationship between water hardness and deicer toxicity. A report was published in 2009 on detailed work completed by the Iowa Department of Natural Resources (DNR) and US EPA, in which toxicity data on the water flea (*Ceriodaphnia dubia*), planorbid snail (*Gyraulus parvus*), tubificid worm (*Tubifex tubifex*), and fingernail clam (*Sphaerium simile*) revealed a relationship between chloride toxicity, water hardness, and sulfate concentrations. They developed a set of criterion equations rather than values for chloride that considered water hardness and sulfate concentration. The developed equations for acute chloride criteria = $287.8\times[\text{hardness}]^{0.205797}\times[\text{sulfate}]^{-0.07452}$ and the equation for chronic chloride criteria = $177.87\times[\text{hardness}]^{0.205797}\times[\text{sulfate}]^{-0.07452}$. This approach to assess chloride toxicity is better able to provide accurate toxicity results for site specific locations (Iowa DNR, 2009). Findings from this testing are significant as they show the need for measuring water hardness and sulfate concentrations during toxicity testing. Water hardness and sulfate values can be used to re-calculate acute and chronic chloride toxicity values. The calculated acute and chronic chloride toxicity values can then be used in the analysis process.

The following equations were developed for calculating acute and chronic criteria based on the influence of site specific water hardness and sulfate concentrations (Iowa DNR, 2009):

$$\text{Acute Criteria Value (mg/L)} = (254.3)^{\dagger}\,287.8^{\ddagger}\times(\text{WH})^{0.205797}\times(\text{S})^{-0.07452} \quad (1)$$

$$\text{Chronic Criteria Value (mg/L)} = (161.5)^{\dagger}\,177.87^{\ddagger}\times(\text{WH})^{0.205797}\times(\text{S})^{-0.07452} \quad (2)$$

Where, † - Values recommended by Iowa DNR (2009), ‡ - Values used for example equations in MassDOT (2012), WH – Water Hardness (mg/L), and S – Sulfate (mg/L).

4) Sulfates

Sulfate may form salts with sodium, magnesium, potassium, and other cations (Iowa DNR, 2009), and it is widely distributed naturally and may be present in natural waters at concentrations ranging from a few to several hundred mg/L. Sulfate ions were found to buffer against chloride toxicity in work by the Iowa DNR and US EPA (Iowa DNR, 2009). Through calculating sulfate concentration based on water hardness and chloride concentration, the obtained sulfate value, in conjunction with water hardness, can then be plugged into Equations 2 and 3 to recalculate acute and chronic chloride toxicity criteria.

5) Biological oxygen demand

Currently available aircraft deicing and anti-icing fluids are made of either propylene glycol or ethylene glycol. The breakdown of glycol can cause elevated biochemical oxygen demand or BOD (ACRP, 2008). The commonly used products in the United States with the lowest BODs are formulated from potassium acetate (KAc) and sodium formate. Common environmental concerns resulting from BOD in snow and ice control products include low dissolved oxygen (DO) in receiving waters, bacterial growth in receiving waters, and odor associated with the runoff. BOD is determined by incubating a sealed sample of water for five days and measuring the loss of oxygen from the beginning to the end of the test.

6) Alkalinity

Alkaline compounds in the water such as bicarbonates (e.g., baking soda), carbonates, and hydroxides remove hydrogen (H^+) ions and lower the acidity of the water. Without this acid-neutralizing capacity, any acid added to a stream would cause an immediate change in the pH. Measuring alkalinity is important in determining a stream's ability to neutralize acidic pollution from rainfall, runoff, or wastewater. Alkalinity is one of the best measures of the sensitivity of a waterway to acid inputs. For total alkalinity, a double endpoint titration using a pH meter and a digital titrator or buret is recommended. This can be done in the field or in the lab, and if analyzing alkalinity in the field, a digital titrator is recommended over a burette (Godfrey, 1988; APHA, 1992).

7) pH value

pH affects many chemical and biological processes in the water, e.g., different organisms flourish within different ranges of pH. Most unimpaired aquatic systems have a pH range of 6.5 to 8.0, and most aquatic organisms prefer this pH range. Outside this range, the diversity of aquatic organisms is reduced due to stress on physiological systems of most organisms and reduced reproduction. Low pH can also allow toxic elements and compounds to become mobile and "available" for uptake by aquatic organisms. pH can be analyzed in the field or in the lab. If it is analyzed in the lab, the pH must be measured within 2 hours of the sample collection due to the effect of carbon dioxide. If a high degree of accuracy and precision is required, pH should be measured with a laboratory quality pH meter and electrode (APHA, 1992; Godfrey, 1988).

Field testing. ***Field sample collection options.*** Water sample collection from the field can be done in two ways using grab samples and composite samples (US EPA, 2002b). Grab samples are easy to collect with a minimal amount of equipment and time, and provide a measure of instantaneous toxicity, but they only reflect a short period of time and are often collected on an infrequent basis. The chances of detecting a spike in contaminant concentration would depend on sampling frequency. Composite samples are collected over a longer time period (24-hr period or designated time period), and better capture concentrations occurring over that time and are more likely to detect spikes in concentration, but collecting these samples is harder, more expensive, and time intensive. Trowbridge et al. (2010) found that monthly grab samples is a sufficient measure of annual average chloride concentration. It would be appropriate where high chloride concentration groundwater recharges waters during times of low flow, or where groundwater flow dominates, but it would not be good in areas where runoff, or melt water, is an issue.

For rivers and streams, samples should be collected mid-stream at mid-depth, and for lakes at mid-depth (US EPA, 2002a). Once collected, effluent and receiving water samples should be handled, preserved, and shipped according to US EPA guidelines (2002b). For testing of deicing toxicity on fresh water organisms, there are two options: 1) using water samples collected in the field, and 2) in-situ monitoring of chloride concentrations and then creating synthetic testing solution with chloride concentration based on those observed in the field measurements.

Selection of a sampling location can be based on water bodies' location, what the water body contributes to, the land-use within proximity to the water way, identified problem areas, or ecologically sensitive areas. Selection of sampling sites that have had historic monitoring or that have a USGS gauging station will aid in long term analysis. Sampling periods can target runoff events during road-salt application periods. For work completed by Corsi et al. (2010), new testing was completed during winter months, and historic data was used to look at year round, summer and winter, specific conductance, to determine historic chloride concentrations.

Timing of sampling. Field sample timing may vary depending on the goals of the research. When designing a sampling plan, there is a need to consider the sampling environment, species of interest, and the life stage of those species. For example, amphibians reproduce in early spring (March – May) in North Carolina, therefore it is important to analyze snowfall and the associated salt load (determined from salt usage data) during this time frame to fully assess impacts to their larval growth stages (Winston et al. 2012). The sampling following a major flushing event could generally capture the highest potential concentration of contaminants seen in the environment (Silver et al., 2009).

Variables to consider for testing. At a minimum the US EPA (2002a) recommends testing effluent or receiving water for PH, conductivity, total residual chlorine, and dilution water for pH and conductivity, and recommends testing total alkalinity and total hardness. Based on work by the Iowa DNR and US EPA, additional testing of sulfate concentrations is recommended, in conjunction with water hardness testing (Iowa DNR, 2009). In the initial sampling, testing for heavy metals and poly aromatic hydrocarbons (PAH) should be considered. Weather data collected from NOAA, RWIS, SNOWTEL, NWS, airports, etc. will aid in the final analysis, including air temperature (maximum and mean), precipitation type and amount, and calculated water equivalent of snow. Information on winter maintenance operations performed throughout the year that may impact your field site should be recorded.

Bio-assessment. To complement the laboratory and field toxicity tests, bio-monitoring can be used to assess the ecological condition of a water way using biological indicators, or biocriteria (Yoder and Rankin, 1998; Gerhardt, 1999; Adams, 2002). Biocriteria can take into consideration physical, chemical and biological stressors and the effects on organisms, spatially and temporally. Biological variables that can be tested include (Gerhardt, 1999): toxicity (organisms' response to chemicals at various organizational levels using bioassays or early warning detection systems) and ecosystem response (assessment of ecological integrity).

Essentially, biocriteria allow for a more complete assessment of toxicity of a product to an ecosystem, including impacts to the food web and lifecycle. Two general methods used in bio-monitoring include: (i) testing *autoecological* parameters using ecotoxicological bio-monitoring in the lab, in-situ, or using online bio-monitoring, or (ii) testing *synecological* parameters, looking at long-term trends using bio-monitoring in the field (Gerhardt, 1999). Another method that can be used to assess chloride toxicity is the Chloride Contamination Index (CCI) developed by Williams et al. (2000) to assess the response of macroinvertebrates to rising salt concentrations in groundwater springs from deicing products.

FUTURE WORK

Currently, the acute and chronic laboratory toxicity test methods determine toxicity for various life-stages of organisms, but research is needed in the area of the impacts of chloride-based chemicals on the ecology of the water body and impacts to food web dynamics. To gain better insights into the role of roadway deicers on stream ecology, field monitoring of aquatic species and populations, bio-assessments or bio-monitoring over time, and continuous monitoring of the environmental parameters in water bodies should be considered in field toxicity testing. Biological monitoring allows for a direct assessment of the aquatic biota and directly measures the biological health of the waterway.

In addition, the results obtained by the Iowa DNR reveal the complexities of toxicity measurements in terms of field assessments. Many variables need to be taken into consideration for accurate field toxicity measurements. Therefore, to implement a reliable field monitoring program, it is essential to determine specific parameters of concern and develop effective monitoring strategies to understand the potential toxicity in a field scenario. Research is needed to accurately correlate laboratory toxicity data with field data. Additionally, more efforts are needed to explore the effects of other types of deicers, including some deicers used in specific areas, and better understand the interactions of various roadway deicers and how parameters such as water hardness, loading rates, hydrologic conditions, land use, time of year, temperature, snow and melting events, and concentrations of other constituents present in waterways affects the toxicity of roadway deicers to aquatic species. Continuous field monitoring programs will provide winter maintenance agencies real-time data throughout the year, which can assist winter maintenance agencies with decision making with respect to application rates in specific areas or types of deicers used in order to reduce the impacts of continued loading of chloride deicers especially in more sensitive areas. Table 1 summarizes some of the critical variables to consider when implementing a field water monitoring program.

Table 1. Variables to consider when implementing a field water monitoring program.

Field Variables to Consider	Significance
Water Temperature	Can have impacts on toxicity due to direct relationship with solubility of chloride salts.
Flow Rate	High flow rates can provide high dilution rates thus reducing concentrations of roadway deicers. Melting events can increase chloride loading.
pH	Affects many chemical and biological processes.
Water Hardness	Increased water hardness decreases toxicity of chloride based roadway deicers (Iowa DNR, 2009).
Sulfates	Increased levels of sulfates decreases toxicity of chloride base roadway deicers (Iowa DNR, 2009)
Timing of Sampling	Recommended to sample during flushing event to capture peak concentrations and determine residence time of deicers.
Bio-monitoring	Assess ecological conditions of water way and determine toxicity impacts.

CONCLUDING REMARKS

This work has presented a state-of-the-knowledge review on the negative/toxicological effects of chloride-based chemicals on the natural environment, and discussed common methods to assess the toxicity of chloride chemicals to water body and aquatic biota in the laboratory and field tests. In detail, species to test, calculation process, and influential factors in the laboratory test and the field sample collection options, timing of sampling, key variables, and bio-assessments in the field test were examined.

Some key findings include The US EPA has utilized aquatic toxicity tests of pure chemicals to establish current water quality criteria. Suggested testing practices include using the static-renewal method or a flow-through system. Continuous real-time *in situ* monitoring of temperature, salinity specific conductivity, pH, water hardness, and DO will provide the most complete data set for assessing the presence and impacts of chloride deicers in a water body. Sampling techniques used may vary based on desired outcomes; monthly grab samples may be used to assess average chloride concentration, but to capture chloride concentration spikes associated with application of deicers and runoff events, hourly grab samples or 24-hr composite/continuous sampling can be used.

Based on the identification the designated use of a waterway, adopting a "Use Attainability Analysis" is one method to potentially justify whether an effluent limit is reasonably attainable or if a more or less stringent effluent limit should apply. Work conducted by the Iowa Department of Natural Resources and US EPA found that some species were more sensitive to short term, acute exposures of elevated chloride concentrations and recommended a lower acute criterion of 682 mg/L (instead of 860 mg/L), but a higher chronic criterion of 406 mg/L (instead of 230 mg/L), based on the Acute to Chronic Ratio (ACR) for the more sensitive species tested (Iowa DNR, 2009). It was also found that increased water hardness and sulfate ions buffer against potential toxic effects of chloride. The results of the Iowa DNR and US EPA study can be used by other states to set site specific acute and chronic chloride criteria based on measured water hardness and sulfate levels. This criterion has been adopted by Iowa and the US EPA Region 7, as well as many more states. The new chloride criteria does not change regulatory requirements for drinking water, but does provide site-specific criteria for aquatic life protection that is based on local water hardness and sulfate levels. This may impact effluent permits and will allow permittees to request modifications to the current permits based on US EPA approval of the new chloride criteria.

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Water Quality Implications and the Toxicological Effects of Chloride Based Deicers

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Abstract

Traditionally, the main priority of winter road maintenance has been assigned to level of service, cost-effectiveness, and corrosion rather than other less well-characterized effects such as impacts to water quality. It is increasingly vital to understand the environmental footprint of deicers, including their impacts on aquatic ecosystems. Chloride based deicers do not degrade in the natural environment and their application on winter pavements can lead to accumulation in adjacent environments over time. Information presented to date on deicers generally includes chemical composition and performance of deicers, while additional information on deicer aquatic toxicity is needed to enable fully-informed decisions by stakeholders. This work presents a state-of-the-knowledge review of the impacts of chloride based deicers and additives on water and aquatic species, and the issue of heavy metal leaching. Toxicity associated with the direct effect of deicers or with the indirect effect via their interactions with runoff chemistry is reviewed as well. This work will assist the stakeholder agencies in the search for effective practices to reduce the toxicity and other water quality implications of chloride based deicers.

INTRODUCTION

Chloride based deicers are well known as essential tools for increasing the safety and mobility of winter roadways (Andrey, 2010; Strong et al., 2010; Usman et al., 2010; Shahdah and Fu, 2010; Ye et al., 2013). Increasing contamination derived from the continued use of deicers has become a significant environmental concern as detrimental effects on water, soil, vegetation, and wildlife have been observed in the field and a variety of risks have been assessed in the laboratory (Ramakrishna and Viraraghavan, 2005; Levelton Consultants Limited, 2007; Fay and Shi, 2012; Wang et al., 2012; Fay et al., 2013a, 2013b; Ke et al., 2013; Cooper et al., 2014; Dudley et al., 2014; Dailey et al., 2014; Baek et al., 2014; Devitt et al., 2014). Of particular interest are the toxicological effects of chloride based deicers in the natural environment. Specifically, the chloride based deicers may present cumulative risk over time, as they are highly mobile (migrating quickly in the environment) and conservative (non-degradable). It is thus difficult to remove them from environment (Environment Canada, 1999; Mason et al., 1999; Kaushall et al., 2005; Winston et al., 2012). The impetus for this work was the assessment of water bodies by US Environmental Protection Agency (EPA) and the designation of some of these water bodies as "impaired" because of chlorides and their potential effects on stream biota (Keller and Cavallaro, 2008). This has in part led to the establishment of total maximum daily loads (TMDL) requirements restricting departments of transportation (DOTs)' use of chloride based deicers.

RESEARCH OBJECTIVES

The objectives of this work are to synthesize the available information in the published domain on the increasingly important water quality issue associated with chloride based deicers and then use this information to assist agencies in developing guidelines, standards and practices to protect water resources from this risk.

BACKGROUND

Since the 1960s, chloride salts have been the primary products used by roadway agencies for deicing (breaking the ice-pavement bond, typically by application of solid chemical after the bond formation). More recently, they are also used for pre-wetting (addition of chemicals to solid salt or abrasives) and anti-icing (prevention of the ice-pavement bond, typically by application of liquid chemical in advance of the bond formation). Road salts can enter the roadside environment through a wide variety of pathways, such as runoff, splash and spray from vehicles, or from snow that has been plowed or hauled off site (Zinger and Delisle, 1988; Ramakrishna and Viraraghavan, 2005). Figure 1 outlines the pathways of deicer movement in the environment (Rubin et al., 2010; Ratkevičius et al., 2014). Once applied, deicers can become a part of road runoff or be dispersed aerially. Once in runoff, they can be part of a surface flow (40-50%) or groundwater transportation (50-60%) and therefore impact soil, flora and fauna. If distributed aerially, deicers can impact the soil, water (mostly surface water), flora, and air quality (Rubin et al., 2010; Ratkevičius et al., 2014; Cheng and Guthrie, 1998; Fischel, 2001).

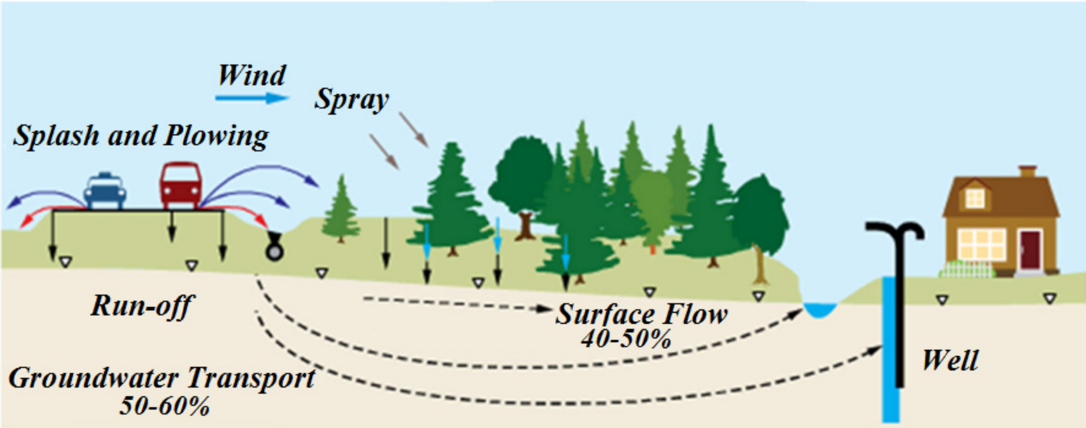

Figure 1. Pathways of deicers into the environment (adapted from Rubin et al., 2010; Ratkevičius et al., 2014).

Information presented to date on the aquatic toxicity of chloride based deicers generally includes chemical composition and properties of deicers. Additional information on their aquatic toxicity such as impacts of chlorides on aquatic species is needed to enable fully-informed decisions by stakeholders. Such toxicity could be associated with the direct effect of deicers or with the indirect effect via their interactions with runoff chemistry. Toxicity testing has not been required of manufacturers or vendors producing chloride based deicers and the associated additives (e.g., corrosion inhibitors and thickeners). As such, little toxicity data has been collected or published in the field of winter maintenance (Corsi et al., 2010). Toxicity research has been conducted on similar products used in other fields including aircraft or airfield deicers (Corsi et al., 2006) and dust suppressants (Williams and Little, 2011), but available data are still very limited. For roadway winter operations, deicers tend to have limited inclusion of additives, which are generally more toxic component compared to the aircraft or airfield deicers (Pillard, 1995; Corsi et al., 2009). Roadway deicers may have additives such as anti-caking agents, corrosion inhibitors, dyes, beet juice, molasses, corn, and other agricultural products (Levelton Consultants Limited, 2007; Fay and Shi, 2012).

Concerns associated with the use of chloride based deicers include increased salinity in waterways and soils, infiltration of cations (e.g., Na^+, Ca^{2+}, and Mg^{2+}) and chloride anion (Cl^-) into soils and drinking water, degradation of the environment along the roadside, and potential risk to biological diversity (TRB, 1991a; Environment Canada, 2001). Studies have demonstrated that sodium chloride (NaCl) can pose significant risks to vegetation, soil, water, and aquatic life and mammals (Vitaliano, 1992; Robidoux and Delisle, 2001; Fay and Shi, 2012). Since the 1970s, road salts have also been shown to impact flora and fauna in the near-road environment, including water bodies (Crowther and Hynes, 1977; Dickman and Gochnauer, 1978). According to Environment Canada (2010), "increased salt concentrations in lakes can lead to stratification, which retards or prevents the seasonal mixing of waters, thereby affecting the distribution of oxygen and nutrients." Bowen and Hinton (1998) found elevated chloride concentrations in surface water affected the spatial patterns of total phosphorous concentrations and induced higher turbidity and bacteria growth, degrading the overall health of streams and making them potentially inhospitable for fish populations. Field research has found that elevated background and spike chloride concentrations in waterways have reduced the richness of amphibian species in some waterways (Turtle, 2000; Houlahan and Findlay, 2003; Collins and Russell, 2009). Deicers have been found to negatively impact aquatic community structures, such that elevated chloride concentration are correlated with reduced species richness and food web dynamics (Sanzo and Hecnar, 2006; Collins and Russell, 2009; Van Meter et al., 2011). Laboratory and field studies in Canada found that in southern Ontario, Quebec and other areas of heavy road salt use, chloride concentrations in ground and surface water are often sufficient to affect biota (Environment Canada, 2001). While changes in macroinvertebrate drift behavior and mortality caused by elevated salt concentrations were found to be highly dependent on concentrations and exposure time, and vary among taxa (Blasius and Merritt, 2002).

Work by Granato (1996) found that in addition to adding the chemical constituents (Na^+, Ca^+, and Cl^-) to aquatic systems, the use of NaCl and $CaCl_2$ deicers can also be considered a source of constituents in runoff:

- NaCl deicer – sulfate, calcium, potassium, bromide, vanadium, magnesium, and fluoride
- Calcium chloride ($CaCl_2$) deicer - sodium, potassium, sulfate, bromide, silica, fluoride, strontium, magnesium

This work did not list potential constituents associated with the use of $MgCl_2$ based deicers.

Chloride salts are readily soluble in water, difficult to remove, and tend to accumulate over time (Howard and Haynes, 1993). Chloride anions are highly mobile, do not "significantly absorb into mineral surfaces", and do not "biodegrade, volatilize, (or) easily precipitate" (Bowen and Hinton, 1998). It has been shown that a high percentage of deicers can migrate from the road as surface runoff and then enter streams and rivers (Crowther and Hynes, 1977; Scott, 1981; Hoffman et al., 1981; Demers and Sage, 1990). Evidence shows that chloride salts can accumulate in aquatic systems, and are conservative (Environment Canada, 1999; Mason et al., 1999; Kaushall et al., 2005; Winston et al., 2012). According to Environment Canada (2001), "research has shown that 10-60% of the salt applied enters shallow subsurface waters and accumulates until steady-state concentrations are attained." Road salt applied by the Swedish National Road Administration contributed to more than half of the total chloride load for the river basin of Sagan (Thunqvist, 2004). Dailey et al. (2014) sampled central Ohio rivers in 2012-2013 and "high Cl^-/Br^- mass ratios in the Ohio surface waters indicated the source of Cl^-" was in part road salt. They also revealed "increasing trends in Cl^- and Na^+ concentration beginning in the 1960s at river locations with more complete historical datasets."

Elevated chloride concentrations are generally linked to spring flushing events. Of special importance is the study of snowmelt runoff from the first major snowmelt due to high concentrations of pollutants contained in this "first flush". A field study in Ohio (Dailey et al., 2014) also observed "higher Cl^- and Na^+ concentrations and fluxes…in late winter as a result of increased road salt application during winter months". Elevated chloride concentrations have been observed in summer, due to recharge of surface water by ground water during times of low flow (Environment Canada, 2001). Shallow wells, reservoirs, and low-flow surface waters adjacent to roadways or storage sites are most susceptible to contamination by deicers as they can infiltrate into groundwater aquifers (TRB, 1991a). The Pennsylvania DOT and some other northern state DOTs have had to purchase wells or provide replacement water where contamination has occurred (TRB, 1991a).

Current standards used by US EPA for chloride in surface waters are 230 mg/L maximum chronic exposure for a four-day average concentration of dissolved chloride, associated with sodium, and 860 mg/L maximum acute exposure for a one-hour average concentration, no more than once every three years on the average [US EPA 440/5-88-001)] (US EPA, 1988). The US EPA standard should apply to most locations except where more conservative values may be necessary, such as where locally important and sensitive species are present. When chloride concentrations in streams reach or exceed the acute and chronic standards, toxicity to aquatic species can result. In addition to toxicological impacts, elevated chloride concentrations can disrupt normal functions of surface water; e.g., cause density stratification and late vertical mixing in waters. A state can adopt these standards or develop a site-specific or pollutant-specific standard if it is adequately justified, which can be higher or lower than the federal standard. Work completed by the Iowa DNR and US EPA can be used as the basis to set site-specific acute and chronic chloride criteria based on measured water hardness and sulfate levels (Iowa DNR, 2009). The established equations are provided as follows: acute chloride threshold = $287.8\times[\text{hardness}]^{0.205797}\times[\text{sulfate}]^{-0.07452}$ and chronic chloride threshold = $177.87\times[\text{hardness}]^{0.205797}\times[\text{sulfate}]^{-0.07452}$. The new chloride criteria "appear to provide a more accurate representation of the potential toxic effects of elevated chloride levels in freshwater streams and rivers" (MassDOT, 2012).

Impaired or threatened waters (rivers segments, streams, lakes) identified by states, where the required pollution controls are not enough to maintain or reach applicable water quality standards are added to the 303(d) list maintained by the US EPA, and will have TMDLs established based on the severity of the pollution and the sensitivity of the waters (US EPA, 2012). The 977 waterways currently considered impaired or threatened waterways are on 303(d) listings due to impacts associated with elevated salinity, chlorides, and total dissolved solids (TDS). The following parameters that are of interest to chloride toxicity and are covered by the 303(d) listing include chlorides, TDS, total suspended sediments (TSS), and biological oxygen demand (BOD). In 2004, New Hampshire added five waterways along I-93 to the list of chloride impaired, reaching a total of nine streams violating the states standard for chloride, after elevated ground water chloride levels were detected, associated with years of heavy salt use (Fredrick and Goo, 2006). Cooper et al. (2014) mentioned that groundwater is a "long-term reservoir for accumulating road salts" and as a result, these accumulated "road salts represent a risk to the safety of drinking water sources." The mechanism proposed by Corsi et al. (2015) suggests that during the snowfall seasons, chloride is stored in shallow groundwater systems and after that is slowly released into the baseflow.

WATER QUALITY IMPACTS BY CHLORIDE DEICERS AND ADDITIVES ECOSYSTEM HEALTH IMPACTS OF CHLORINE DEICERS AND ADDITIVES

Chlorides are readily soluble in water and difficult to remove, and thus concerns have been raised over their short-term and long-term risks to water quality, on aquatic organisms, and on human health (TRB, 1991b; Environment Canada, 2010). The chloride salts applied on winter roads can migrate into nearby surface waters and impact them via various pathways. NaCl from road runoff is responsible for increased salinity or osmolality of surface and ground waters several months after the last road treatment (Thunqvist, 2004). Ramakrishna and Viraraghavatan (2005) reported that this could reduce water circulation and re-aeration in lower depths, resulting in changes in population or community structure of aquatic biota.

Impacts of chloride based deicers on water. Karraker et al. (2008) investigated the effects of road salt on water quality, by measuring water quality variables at 28 roadside pools and 14 forest pools. Water temperature, pH, dissolved oxygen, and conductivity in each pool were measured monthly May through August. The results showed that the conductivity, pH, dissolved oxygen, and water temperature differed between forest and roadside pools. Mean conductivity was nearly 20 times higher in roadside (357.8 μS; range 11.6 – 2904.8 μS) than forest pools (18.6 μS; range 5.7 – 41.4 μS). Conductivity was strongly correlated with both sodium and chloride. Sixty-one percent of roadside pools had higher average conductivity than all forest pools. Conductivity in roadside pools declined exponentially with increasing distance from road (Karraker et al., 2008). Mean water temperature was lower in roadside (48.9°F, 9.38°C) than forest (55.6°F, 13.1°C) pools. Mean pH was higher in roadside (5.3) than forest (4.7) pools. On the other hand, the dissolved oxygen (mg/L) was nearly one-third higher in roadside pools (3.6 mg/L) than in forest pools (2.0 mg/L). The results also showed that water quality variables varied over time. Conductivity, pH, and water temperature increased between May and August in both roadside and forest pools. Conductivity and dissolved oxygen in pools were similar in May among years. However, pH decreased and temperature decreased. Forest ponds were larger in perimeter than roadside ponds, but similar in maximum pond depth and canopy cover (Karraker et al., 2008).

Deicer impacts on surface and ground waters depend on the site-specific properties of the receiving water body. The quantity of precipitation affects the dilution rate of the applied deicers and the flushing rate of the system (TRB, 1991a). Factors unique to each site can also influence potential impacts – temperature, topography, sunlight, wind, etc. Additionally, surface waters have a wide array of physical, biological, and chemical cycles and interacting processes (Mayer et al., 1999). Field testing of chlorides has produced widely variable results from no observed effects (Jones et al. 1992; Baroga, 2005), to highly elevated chloride concentrations (TRB, 1991a; Godwin et al., 2005; Environment Canada, 2010). It is very important to understand how the size and flow rate of the water body influence the impacts of chloride contamination on aquatic species. Questions remain on: how do cold temperatures or other seasonal factors affect the tolerance on aquatic species, before seasonal standards or criteria can be established; what are the appropriate location and frequency of water quality sampling for determining representative chloride concentration non-compliance due to deicer applications; what are the guidelines for optimizing the potential of road right-of-way in mitigating the flow rate and composition of chloride-laden stormwater runoff. These issues should be explored by future studies.

Impacts of chloride based deicers on aquatic species. Salt has negative effects on aquatic organisms by altering the osmotic balance between the organism and its surrounding environment. Salt concentrations less than those of the organism's protoplasm or other fluids are common in freshwaters and most terrestrial soils, and so the organisms must expend energy to prevent simple diffusion of salts out of their bodies (Findlay and Kelly, 2011). If concentrations rise to surpass the isotonic point of the organism, it faces the converse problem of an inwardly directed diffusion gradient and must either exclude ions or actively excrete those that may cross their integument/membrane. Threshold levels for salt effects vary widely across organisms and/or life stages (Findlay and Kelly, 2011).

Many studies have addressed the effects of chloride based deicers on wildlife species and research has shown that road salt exposure negatively affects mammals, birds, invertebrates, and amphibians that utilize roadside habitats (Vitt et al., 1990; Mineau and Brownlee, 2005; Ramakrishna and Viraraghavan, 2005; Harless et al., 2011; Karraker and Gibbs, 2011). However, most field data are correlative, where investigators survey a range of water bodies that may vary in salt loads but undoubtedly also vary in other contaminants (Peters, 2009). In a survey of macroinvertebrates in urban streams, Cuffney et al. (2010) found that the chloride ion was frequently and strongly associated with indicators of poor macroinvertebrate communities, but it was acknowledged that other measured (and probably unmeasured) solutes co-varied with chloride, which may have included stream geomorphology, habitat availability, metals or organic compounds associated with urban runoff or wastewaters. Furthermore, Corsi et al. (2010) found road salt to cause detrimental impacts to surface water on local, regional, and national scales, with short- and long-term impacts to stream water quality and aquatic life.

Fay et al. (2013a) cited that by increasing the volume of water or the flow rate of stream, the tolerability towards deicers will increase which is due to the diluted incoming chloride. In general, the degree to which the surface water is contaminated from deicers is a function of the amount of time the deicer takes to reach the water body, the dilution factor, the residence time of the water body, and the frequency and rate of deicer application (D'Itri, 1992).

Amphibian toxicity. The deposition of chemical pollutants into roadside wetlands from runoff is a current environmental concern. In northern latitudes, a major pollutant in runoff water is NaCl from deicers. Collins and Russell (2009) conducted a survey from 26 roadside ponds for amphibian species richness as compared to chloride concentration and concluded that chloride concentrations in ponds associated with deicing salts, influenced community structure by excluding salt intolerant species. Many studies have found that as salinity increases survivorship and mobility decrease (Sanzo and Hecnar, 2006; Karraker and Ruthig, 2009; Denoël et al., 2010). Amphibians are likely to be the most affected by chemical and deicer runoff. Amphibians possess highly permeable skin and have aquatic larval stages, and many use roadside wetlands for breeding (Harless et al., 2011). Amphibians are considered good indicators of ecosystem health, and inhabit a wide range of habitats (wetlands, ephemeral water bodies, road side ditches, etc.) and for these reasons extensive toxicity testing has been conducted using amphibians (Vitt et al., 1990). Of the amphibian species tested, salamanders may be the most intolerant amphibians to varying salt concentrations (Collins and Russell, 2009).

Road salts and heavy metals have been shown to have the greatest impacts on amphibians (Birdsall et al., 1986; Sanzo and Hecnar, 2006). Studies have found that chloride concentration spikes that occur during spring runoff and from direct runoff from the road threaten embryonic and larval stage of amphibians (Collins and Russell, 2009). The effects of hyposalinity on the wood frog (Rana sylvatica) included a significant decrease in body weight, not undergoing metamorphosis, decreased activity and increased developmental abnormalities, and death (Sanzo and Hecnar, 2006). Kim and Koretsky (2013) collected sediment cores in fall and spring from a freshwater wetland fringing an urban kettle lake in Michigan and "incubated for 100 days in deionized water (control) or with treatments of 1 or 5 g/L $CaCl_2.2H_2O$ or 5 g/L NaCl to simulate addition of road salt deicers". Their study revealed that the influx of chloride deicers would significantly affect the microbial activities in wetland and biogeochemistry of wetland sediments. In contrast, a cold-climate outdoor study of bioretention mesocosms with soil, mulch and vegetation layers (Denich et al., 2013) indicated that the application of a salt/aggregate mixture did not significantly affect the mobility of heavy metals in the simulated bioretention facility.

Even between amphibian species, toxicity is highly variable. For example road salt were found to significantly affect embryonic survival of the spotted salamander (Ambystoma maculatum), while having little effect on the green frog (Rana clamitans) (Karraker and Ruthig, 2009). In a study conducted by Collins and Russell (2009), it was found that spotted salamander (Ambystoma maculatum) and wood frogs (Rana sylvatica) did not occupy high chloride ponds, and acute toxicity tests (LC50) found them to be more sensitive than the American toad (Bufo americanus) to chloride. Spring peepers (Pseudacris crucifer) and green frogs (Rana clamitans) showed intermediate sensitivities (Table 1).

A case study in North Carolina on the impacts of deicers to amphibians found that the amount of snow, or water equivalent, is important because for many DOTs the application rate is relatively constant (250 lbs/l-m) per snow or ice event (Winston et al., 2012). Therefore smaller snow and runoff events can have a greater impact on amphibian populations than larger snowfalls.

Table 1. A summary of example different toxicity studies including chemicals used, species tested and the observed LC50 values at 48 h and 96 h of exposure to chemical deicers.

Reference	*Chemical used*	*Species tested*	*LC50 (g/L Cl^-)*	
			48-h	*96-h*
Collins and Russell (2009)	NaCl	Spotted salamander		1.18
		Wood frog		1.72
		Spring peepers		2.83
		Green frogs		3.11
		American toads		3.93
Harless et al. (2011)	CH_4N_2O	Wood frog	14.37	14.29
	NaCl	Wood frog	7.82	7.56
	$MgCl_2$	Wood frog	7.28	7.11
	CH_3COOK	Wood frog	5.42	4.23
	$CaCl_2$	Wood frog	4.72	3.98
	$C_8H_{12}CaMgO_8$	Wood frog	3.39	3.23
Baek (2014)	$CaCl_2$	Cloeon dipterum	6.14	
	$CaCl_2$ (Deicer)	Cloeon dipterum	6.50	
	$CaCl_2$	Ecdyonurus levis	6.32	
	$CaCl_2$ (Deicer)	Ecdyonurus levis	8.34	
	$CaCl_2$	Glyptotendipes tokunagai	3.84	
	$CaCl_2$ (Deicer)	Glyptotendipes tokunagai	5.69	
	$CaCl_2$	Gammarus sobaegensis	3.54	
	$CaCl_2$ (Deicer)	Gammarus sobaegensis	5.85	
	$CaCl_2$	Caridina denticulata denticulata	18.88	
	$CaCl_2$ (Deicer)	Caridina denticulata denticulata	20.73	

A study looked six commonly used deicers (urea, NaCl, $CaCl_2$, $MgCl_2$, KAc, and CMA) and their toxicity to larval wood frogs (Rana sylvatica). Survival was inversely related to higher concentrations for all deicers tested (Harless et al., 2011). Tadpole survival had significantly lower threshold concentrations for all deicers. Acetate based deicers had lethal effects on tadpoles at the lowest concentrations. $CaCl_2$ and $MgCl_2$ were found to be more toxic to frog tadpoles than NaCl, most likely because they have two chloride ions per molecule (Dougherty and Smith, 2006; Harless et al., 2011). Interestingly, LC50 values decreased with time, suggesting that organisms were either less capable of tolerating deicers overtime, or that there was a lag in the lethal effect (Harless et al., 2011). The relative toxicity scale for larval and tadpole frogs was: NaCl$<MgCl_2<CaCl_2<$Acetate based (Table 1) (Harless et al., 2011).

Benthic species toxicity. Riva-Murray et al. (2002) found that declines in several indices of macroinvertebrate "health" were correlated with higher chloride concentrations, but the absolute levels were so low (<50 mg/L) that it is unlikely that direct toxicity was causing the patterns. Thus, while it is fair to say that chloride is associated with negative effects on benthic insects at low concentrations, other contaminants and stressors are most likely affecting populations as well. This is confirmed by a more recent study by Baek et al. (2014), which reported that "Cl^- concentrations (from $CaCl_2$ deicer) in the field sites (<25 mg/L) were much lower than the LC50 values of five selected macroinvertebrate species (Gammarus sobaegensis, Caridina denticulata denticulata, Glyptotendipes tokunagai, Cloeon dipterum, and Ecdyonurus levis)". The laboratory test revealed that the LC50 value of the five select species ranged from 3.54 to 20.73 g/L.

Experiments focused on altering salt levels in surface waters have been rare but do provide more direct links between salt concentrations and response variables. For example, NaCl was added to reach ambient concentrations of 1,000 mg Cl^-/L in a stream, and resulted in changes in benthic algae and protozoans within one to a few weeks after initiation of the experiment (Dickman and Gochnauer, 1978; Evans and Frick, 2002). In another study, Cl^- was added to levels of 2,165 mg/L, and caused an increase in drift of benthic insects out of the experimental sub-channels placed in an Ontario stream within a few hours (Crowther and Hynes, 1977).

Baek et al. (2014) conducted quantitative field sampling of benthic macroinvertebrates at eight surface water sites exposed to $CaCl_2$ deicer applications in South Korea and reported that "despite the heavy application of road deicers during the snowy season, the deicer may not directly affect benthic macro-invertebrate communities over short time periods." Casey et al. (2014) mentioned that highway deicing chloride seldom can harm fish, and have no significant harmful effect on microfauna and invertebrates.

Invertebrate species toxicity. Research testing the acute toxicity of chloride to four freshwater invertebrate species, including water flea (Ceriodaphnia dubia), fingernail clam (Sphaerium simile), planorbid snail (Gyraulus parvus), and tubificid worm (Tubifex tubifex), was completed under different levels of water hardness (all four species) and sulfate concentrations (C. dubia only) (Linton and Soucek, 2008). Tests with C. dubia acclimated and tested under different levels of total water hardness and sulfate were performed simultaneously by two different laboratories (Linton and Soucek, 2008). Results were comparable. The 48-h LC50 for C. dubia acclimated and exposed to acutely lethal chloride concentrations at 25 to 50 mg/L hardness (i.e., 919 mg Cl^-/L) is approximately half that of C. dubia exposed at 600 to 800 mg/L hardness. Conversely, sulfate over the range of 25-600 mg/L exerted only a small (inverse) effect on chloride toxicity to C. dubia. The mean 48-h LC50 at 25 mg/L for sulfate was approximately 1,356 mg Cl/L, while at 600 mg/L sulfate, it was 1,192 mg Cl^-/L (reduction of 12%). Again, LC50 values between labs were consistent. Ninety-six hour LC50 values for three other freshwater invertebrate species ranged from a low of 740 mg Cl^-/L for S. simile exposed to chloride at 50 mg/L hardness, to a high of 6,008 for T. tubifex exposed to chloride at 200 mg/L hardness. For both these species, increasing the acclimation and dilution of water hardness reduced the acute toxicity of chloride by approximately 1.4 to 1.5 times. Water hardness did not appear to influence the acute toxicity of chloride to the planorbid snail, G. parvus. Acute LC50 values at 50 and 200 mg/L hardness were 3,078 and 3,009 mg Cl/L, respectively. Rank order of sensitivity to acutely lethal chloride at a given water hardness is in the order (most to least): S. simile>C. dubia>G. parvus>T. tubifex. Siegel (2007) tested toxicity thresholds for NaCl and chloride for various species and found freshwater aquatic species react differently to varying exposure concentrations. Based on these findings, invertebrate species appear to be more sensitive to chloride than are vertebrate species, with fathead minnow being the most sensitive species tested (Siegel, 2007).

Fish toxicity. Salt tolerance of aquatic species varies tremendously. Depending on whether fish are fresh or salt water species, fish have been reported to tolerate between 400 and 30,000 mg/L (Siegel, 2007). Road salting and increased salinity can lead to excess growth of salt tolerant species (Siegel, 2007). Interestingly, aquatic species may adapt to increased levels of chloride with time, such that surviving organisms may develop the means by which to handle the osmotic shock imposed by the excess chloride (Mineau and Brownlee, 2005). Furthermore, saltwater species are not vulnerable to anthropogenic sources of NaCl, except for fluctuations of greater than 10%.

There is a wide range in salt concentrations known to have negative effects on organisms, and most of these data come from laboratory exposures under controlled conditions with only one stressor (salt) acting at a time. For many organisms, the short-term lethal concentrations are far above levels seen in the environment, except under extreme conditions (Findlay and Kelly, 2011). For example, adult freshwater fish species do not show lethal effects until concentrations approach tens of grams of salt per liter. Young fishes show negative effects at much lower levels, but even so, these concentrations are roughly equivalent to those in direct road runoff, much higher than concentrations occurring after dilution by water sources. Effects near roads are, of course, more evident than for points further from the actual area of salt application with reasonably sharp declines in concentrations within tens of meters of the road (Albright, 2005; Lax and Peterson, 2009).

Spring rains have been postulated to dilute deicers following input into waterways. One study tested this theory and found spring rains may help when low to moderate chloride concentrations are present, but in situations where water lacks outflow or flushing and chloride concentration reach 1000 mg/L, permanent damage can occur to egg membranes (Karraker and Gibbs, 2011).

Biotoxicity work for the Colorado DOT on the boreal toad tadpole, juvenile rainbow trout, Ceriodaphnia (aquatic invertebrate), and Selenastrum (algea) found that application rates of $MgCl_2$ typically used are highly unlikely to cause or contribute to environmental damage at distances greater than 20 yards for the roadway (Lewis, 1999). Work in Chautauqua Lake, New York on sunfish found that NaCl contributed to toxicity but zinc and cadmium were also found to be of concern (Adams-Kszos et al., 1990). A study in Washington observed deicer impacts on Steelhead (Oncorhynchus tshawytscha), Chinook salmon (Oncorhynchus mykiss), and bull trout (Salvelinus confluentus), threatened and endangered species, and general stream ecology (Yonge and Marcoe, 2001). Elevated chloride concentrations from 5 to 8 mg/L were found, but the authors suggested the deicing activities did not adversely impact the stream ecology.

Models have shown that cyanide has potential to impact some species in areas of high road salt use (Environment Canada, 2001). Sodium ferrocyanide and ferric ferrocyanide are used to prevent clumping of solid chloride based deicers while in storage and during deicing operations (Letts, 2000). In Canada, sodium ferrocyanide was added to rock salt at a range of 30 to 240 mg/kg of salt. Potassium ferro- and ferricyanide solutions were originally believed to be much less toxic (no effect at 2000 to 8732 mg/kg), but due to a fish kill in a stream caused by ferro- and ferricyanides from industrial effluent, it was found that photo-decomposition can release the cyanide ion (Burdick and Lipschuetz, 1950). Experimental results found ferro- and ferricyanides to be toxic at 1 – 2 mg/kg when photodecomposition was considered (Burdick and Lipschuetz, 1950).

Letts (2000) tested the issue of photodecomposition of ferrocycanide to create free cyanide, a much more toxic substance, in the ecosystems that commonly receive deicer laden runoff. Letts (2000) concluded that in most situations the ferrocyanide will precipitate, break down via photolysis volatilization, or biological degradation before it can cause harm to the ecosystem. Additionally, dilution rates commonly seen in ecosystems receiving deicer laden runoff will reduce the potential for harm to organisms. Letts (2000) does caution that impacts could occur to organisms in roadside ditches in urban areas with heavy salt use. Photoenhancement, or increased aquatic toxicity due to a chemical transformation of a substance due to UV exposure, was also observed by Little and Calfee (2000), when testing the toxicity of sodium ferrocyanide (from yellow prussiate of soda (YPS) used in fire retardant). Rainbow trout (Onchorhyncus mykiss) and Southern leopard frog (Rana sphenocephala) were exposed to various fire retardants and UV light treatments. Rainbow trout were always more sensitive to fire retardants than the Southern leopard frog, and both species were equally affected by low concentration of YPS alone when exposed to UV. Cyanide agents can interfere with a fish's gills impeding breathing causing death (MN Pollution Control Agency, 2000). In Minnesota, run-off from the salt piles was reported to contain between 5 and 40 times the amount of free cyanide that is toxic to half of the fish exposed, caused by concentrated runoff from salt piles that were improperly managed at that time (MN Pollution Control Agency, 2000). Environment Canada (2001) concluded that "road salts that contain inorganic chloride salts with or without ferrocyanide salts are 'toxic'."

Aquatic invertebrates' toxicity. In Iowa, the US EPA tested acute toxicity of chloride to four freshwater species, water flea (Ceriodaphnia dubia), fingernail clam (Sphaerium simile), planorbid snail (Gyraulus parvus), and tubificid worm (Tubifex tubifex) under varying concentration of sulfate and levels of water hardness (Linton and Soucek, 2008). The study found that for two species (S. simile & T. tubifex), increasing the acclimation and dilution of water hardness reduced the acute toxicity of chloride by up to 1.5 times, while sulfate was found to negatively impact chloride toxicity by up to 12% for C. dubia.

Research conducted on aircraft and airfield deicers and dust suppressants has laid the foundation for toxicity testing of roadway deicers. Work by Pillard (1995) investigated the toxicity of ethylene and propylene glycol and the associated aircraft deicer additives on Ceriodaphnia dubia and Pimephales promelas. Corsi et al. (2006; 2009) reported aquatic toxicity data for airfield pavement deicers, KAc and Sodium Formate (NaFm) using US EPA tests methods (US EPA 2002a,b), and aquatic organisms Vibrio fischeri, Pimephales promelas, Ceriodaphnia dubia, and Selenastrum capricornutum. The Airport Cooperative Research Program (ACRP) published an extensive document on developing new formulations for aircraft deicers that have lower toxicity and BOD (ACRP, 2008). Aircraft deicers tested were acetate and formate based products and ethylene and propylene glycols. Toxicity testing was conducted using US EPA methods on fathead minnow, daphnia magna, and rainbow trout. Chauhan et al. (2009) reported that airport runway deicers (e.g., acetates) feature a typical Daphnia magna 48-hr LC50 value of 1,000 mg/L and a typical Pimephales promelas (fathead minnows) 96-hr LC50 value of 1,000 mg/L, whereas the more eco-friendly proprietary alternatives featured LC50 values up to 4,875 mg/L. For chronic toxicity, common airport runway deicers had C. dubia IC25 values typically in the range of 400-820 mg/L, and P. promelas LC25 values typically in the range of 180-280 mg/L, whereas the more eco-friendly proprietary alternatives featured IC25 values in the range of 1,100 to 2,600 mg/L.

Aquatic vegetation is similarly vulnerable to the changes in salt concentrations. One algal species has demonstrated extreme sensitivity to exposures to chloride; with concentrations of 71 mg Cl^-/L inhibiting growth and chlorophyll production, while others can tolerate chloride concentrations between 886 and 36,400 mg/L (US EPA, 1988). An increase in chloride may also allow for non-native species to become more predominant. Other aquatic plants exhibit various sensitivities, with growth inhibition observed in desmids at 200 mg Cl^-/L, EC50 equal to 1482 mg Cl^-/L in diatoms, and reduced growth and reproduction at 1820 mg Cl^-/L in angiosperm (US EPA, 1988). Although noteworthy, the sensitivities exhibited by the algae and desmids do not weigh into the final threshold determination because the toxicity tests were not conducted with measured concentrations of chloride, a biologically significant endpoint, and an aquatic plant of consequence in U.S. waters.

Heavy metal leaching by deicers. Research conducted in Sweden found that concentrations of cadmium (Cd), copper (Cu), lead (Pb), and zinc (Zn) in soil adjacent to roadways were related to the use of deicing salts (Backstrom et al., 2004). The methods of mobilization of the heavy metals included ion exchange, lowered pH, chloride complex formation, and possible colloid dispersion. The researchers express concern about mobilization of heavy metals due to the use of NaCl as a deicing product and potential contamination to shallow ground water.

Soil columns leached with NaCl mobilized organic materials and iron oxides. The potential to carry adsorbed heavy metals along with them increases as electrolyte concentrations decrease (Amrhein, 1992; Norrström, 1998). Zinc and cadmium are far more susceptible to changes in pH and become more mobile with increasing acidity (Backstrom, 2004; Amrhien, 1992). Doner (1978) also performed soil column experiments with both chloride and perchlorate salts to test the theory of complexation as a mechanism of metal transport. Perchlorate has an ionic strength equal to chloride but is known to form very weak metal complexes. The results showed 1.1 to 4 times as much movement of cadmium, nickel and copper for chloride salt leachate as compared to the perchlorate leachate. Cadmium tends to be associated with readily leached compounds and is more mobile than other heavy metals when in soils (Norrström, 1998). The addition of chloride from deicer salts increases the mobility of cadmium via complexation with the chloride. The resulting negatively charged ligand complexes further compete with clay for cadmium ions (Lumsdon et al. 1995). This is also true, to a lesser extent, when CMAs are used by complexation with acetate (Amrheim, 1992). Cadmium has also been demonstrated to be more mobile in high salt concentrations by cation exchange with sodium, magnesium and calcium (Amrhein, 1992; Backstrom, 2004). Chromium is more closely associated with the transport of organic materials and occurs at higher levels when CMAs are used as deicers. This was also true for lead, copper and nickel (Amrhein, 1992).

In the Netherlands, a remediation facility consisting of a detention basin and a constructed wetland were tested for retention of heavy metals and Poly-Aromatic Hydrocarbons (PAHs) from road runoff (Tromp et al., 2012). This study found the system was very effective at removing PAHs, 90-95% removal; however, during application of deicers, concentrations of copper (Cu), zinc, (Zn), cadmium (Cd), and nickel (Ni) were found to be much higher in the wetland effluent. The researchers recommended modifying the hydraulic management of the system, to bypass the road water runoff, during times when deicers are being used so as to maintain the integrity of the remediation facility.

CONCLUDING REMARKS

Environmental risks of chloride based deicers, indicating that the actual effects depend on individual site conditions as well as the type and amount of deicers applied. Overall, maintenance yards have the potential to for releasing high chloride concentration runoff into the adjacent environment and pose a risk of point source pollution. In contrast, stormwater runoff from roads where deicers have been applied at controlled application rates tends to be diluted by precipitation (averaged at 500:1) and poses a risk of non-point source pollution.

Spikes in chloride concentrations in waterways near roads are often observed in the first flushing event, or with spring melting of accumulated snow. If elevated chloride concentrations are observed in times of low flow or when baseflow dominates surface waters near roads, groundwater recharging the waterways may be the source of chlorides.

The density of road networks and the application rates used can directly influence the chloride concentrations observed in surface and groundwater. Land use, soil characteristics and subsurface geology influence groundwater chloride concentrations, as well as precipitation and deicer application rates. Based on these findings, mapping of sensitive areas, road density, annual precipitation, etc. can aid in identifying areas for reduced salt use or where alternative products may be better suited.

Species that have been shown through laboratory testing to be good indicators of the impacts on chlorides are salamanders for amphibious species and fathead minnow. Generally speaking, invertebrate species appear to be more sensitive to the effects of chloride than vertebrate species. Recent laboratory testing has shown that increasing species acclimation to and diluting water hardness reduces the toxic effects of chloride by 1.4 to 1.5 times. The presence of sulfate during chloride toxicity testing was found to increase toxicity by 12% for the same species. This testing was completed with a limited number of species at this point in time.

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