FRAMEWORK FOR RISK-BASED MANAGEMENT AND SAFETY OF RAILROAD BRIDGE INFRASTRUCTURE USING WIRELESS SMART SENSORS (WSS)

BY

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DISSERTATION

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ABSTRACT

To increase overall profitability, add capacity to rail operations to meet projected needs, and comply with new federal regulations on bridge safety, North American railroads are exploring means and methods to improve the management of their bridge networks. Current maintenance, repair, and replacement (MRR) decisions are informed by bridge inspections and ratings. Inspection and rating practices recommend observing the response of bridges under revenue traffic. However, an objective relationship between bridge responses and the impact to railroad operations has yet to be established. Moreover, measuring responses while trains are on the bridge can be quite challenging and sometimes may not be possible. As a result, current MRR decisions are not optimal and in general conservative, prioritizing safety to overcome the uncertainty of consequences of inaction. If the consequences of MRR decisions could be better determined, then the railroads could more effectively allocate their limited resources. This dissertation addresses this issue by developing an approach for consequence-based management of bridge networks, adopted from the field of seismic risk assessment, for making MRR decisions on a network-wide basis. The proposed framework assesses bridge service state condition based on fragility relations. Fragility curves are developed relating bridge responses under revenue service traffic to service condition limit states. Additionally, this research conducted specific Structural Health Monitoring (SHM) campaigns for railroad bridges employing Wireless Smart Sensors (WSS). Wireless strain gages installed in the rail measured real-time trainloads and speeds, while wireless accelerometers and magnetic strain gages measured associated bridge responses. The sensing system was deployed and validated on multiple railroad bridges in North America under different types of traffic and capacity. The measured bridge data can be used to update periodically the fragilities to have more accurate estimates of the bridge condition. The expenses associated with these service

conditions estimate the total costs of a given MRR policy. In this way, MRR decisions can be prioritized minimizing negative consequences to railroad operations. This framework provides a consistent approach for intelligent management of railroad bridges, and more specifically, for the prioritization of railroad bridge MRR decisions. Using this framework the rail owner can identify the most efficient use of a limited budget while maintaining safe railroad operations.

To Conchi, Fernando, Susi, Alex & Cris

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CHAPTER 1 INTRODUCTION

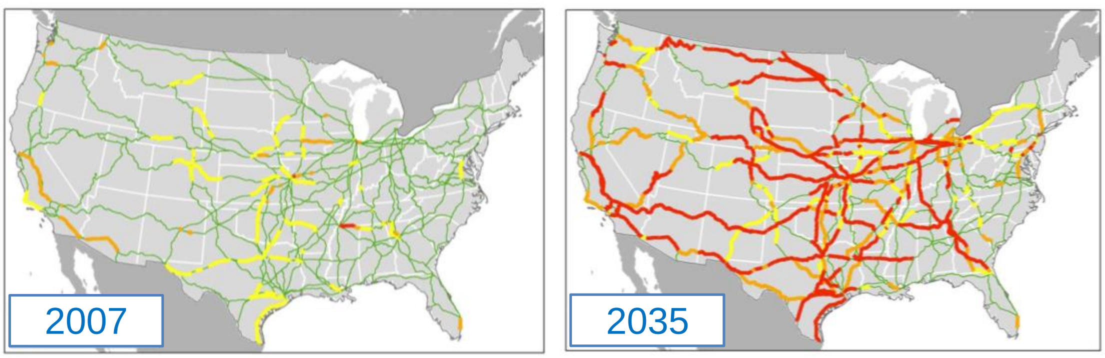
1.1 Importance of US railroads

Rail freight transportation in North America is widely accepted to be the best in the world (GeoMetrx, 2013), with 40% of the nation's freight tonnage transported by train (AAR, 2014). North American railroads expect to exceed their capacities over the next 20 years at many locations within their network (Figure 1) and need to prepare their infrastructure accordingly. One of the largest challenges in creating a financially competitive rail network is to maintain adequate track capacity to address expanding passenger and freight needs. Capacity is the ability of a given railroad to move a given volume of traffic over a specific line under a given Level of Service (LOS) (Lay and Barkan 2009), and it is affected by the maximum operating speed allowed. Railroads in North America have different track classes corresponding to different capacities. Higher track class corresponds to higher speed and higher capacity, see Table 1.1 (FRA, 2015). Railroads in North America have doubled capital investments in the last few decades to meet capacity demands. For example, Class I railroads invested over $12B in capital expenditures in 2012 (Berman, 2012). This investment, combined with technology innovations in freight cars and locomotives, has resulted in a doubling of the average tons of freight per train loading (Weatherford, 2008; Dierkx, 2009). As a result, freight costs per ton-mile have been reduced by roughly 50%, portending that freight carried by North American railroads will increase significantly in the future (Thompson, 2010). In 2012 Amtrak marked its highest year of travelers, with 31.2 million passengers (double the ridership from 2000), and by 2040 expects a 400% increase in passengers in the North East Corridor (ASCE, 2013b). In 2014, railroads spend $26B "to maintain bridges, lay new track, purchase equipment, and upgrade signal systems" (Freight Rail Works, 2015) (Figure 1.2).

Cambridge Systematics, Inc. (2007) estimated the cost of infrastructure expansion needed to match the 2007-2035 projected growth in demand at $148 billion (in 2007 dollars).

Table 1.1. FRA class level and maximum allowable operating speed limits (2015).

Track	Freight Trains	Passenger Trains
Excepted track	10	N/A
Class 1 track	10	15
Class 2 track	25	30
Class 3 track	40	60
Class 4 track	60	80
Class 5 track	80	90



(a) (b)

Level of Service (LOS)	Means	Description
	Below Capacity	Can accommodate maintenance work and recover from incidents
	Near Capacity	Heavy train flow, moderate capacity to accommodate work
	At Capacity	Very heavy train flow
	Above Capacity	Unstable flow; service break-down conditions

Figure 1.1. Railroad capacity levels of service: (a) 2007, (b) 2035 (Cambridge Systematics, 2007).

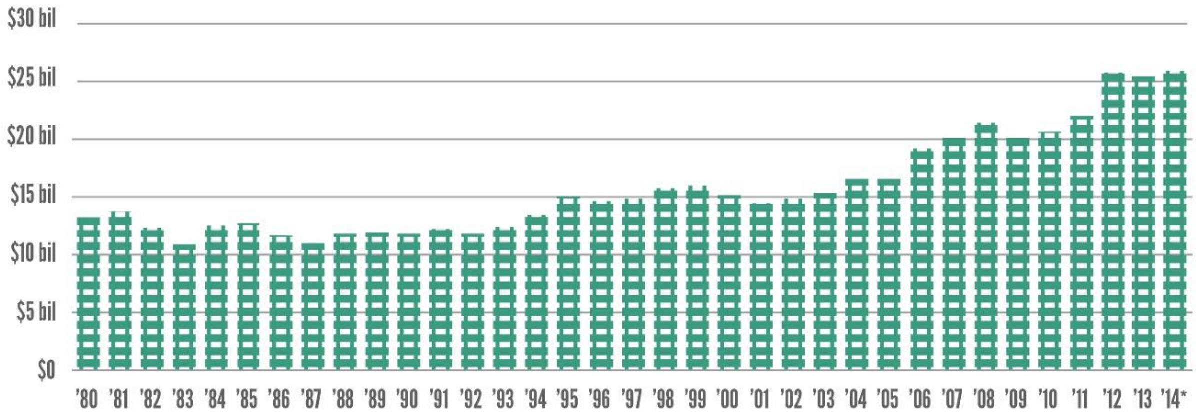


Figure 1.2. Annual rail investment per year (Freight Rail Works, 2015).

The US Federal Government is prioritizing passenger traffic by investing in infrastructure to increase train speeds in shared traffic corridors used by both freight and passenger traffic. In October 2009 President Obama proposed a $5 billion investment towards High Speed Rail (HSR) as part of the 2010 budget (CNN, 2009). In January 27, 2010, the President announced more than $8 billion dollars in funding from the Federal Railroad Administration (FRA) to begin the construction of high-speed railroads (Freemark, 2010). While California envisions building a dedicated track exclusively for the new HSR vision (Schwarzenegger, 2010), the Midwest will host some of the forecasted HSR corridors with both freight and traffic on the same track owned by the freight railroads, according to the Midwest High Speed Rail Association (MSHRA) (2010a, 2010b.) Past negative experiences in the United Kingdom when adopting current track for an upgraded speed shows the risks of adopting higher speeds on existing infrastructure (Carr and Greif, 2000).

1.2 US railroad bridges

Bridges are a critical component of railroad infrastructure, with an average of one bridge for every 1.4 miles of track. The total replacement cost of this railroad bridge inventory is estimated at about $100 billion (Vantuono, 2008). Freight railroad companies own 77,000 railroad bridges in the U.S. (Richards, 2007a; GAO, 2007; FRA, 2008a; FRA, 2008b), and another 1,300 are owned by Amtrak (Cowan, 2004). However, the FRA estimates a total of 100,000 railroad bridges (ENSCO, 1994; FRA, 2008a, 2008b, 2010b), when including commuter, medium, and short railroad companies not typically reached by general inventories in the U.S. Of the 100,000 bridges, a significant portion is approximately 100 years old (Cambridge Systematics, 2007). In particular, the US Department of Transportation reported that more than half were built before 1920 (AREMA, 2003). According to Unsworth (2010), the weight/car has augmented rapidly in the last decades and the capacities are being exceeded in old bridges. Researchers use the term "bridge network" for the transportation network recognizing that the bridge is the most fragile component in the entire system (Bocchini and Frangopol, 2011). Railroad companies need to continuously assess the structural condition (safety) of their bridges to ensure the operational performance of rail networks (Byers and Otter, 2006).

1.3 Maintenance, repair, and replacement (MRR) of US railroad bridges

During the last five years, Class I railroads have consistently invested approximately $500M annually in Maintenance, Repair, and Replacement (MMR) only for bridges, even though this has been a period of general economic recession (Freight Rail Works, 2015), focusing these limited resources on maintaining safe and reliable operation of the network. Each MRR decision is called a "project" decision (for one individual bridge). Work plans group bridges based on the expected time period over which these bridges should be replaced. To determine which bridges would be included in each work plan, railroads use bridge inspection reports (AREMA, 2008, 2014). These inspections have been federally required annually since 2010 (FRA, 2010a). The group of all the individual "project" decisions constitute the "network" decision. To inform which MRR to choose for the MRR of the network, one strategy is choosing bridges from more urgent work plans. Alternatively, railroads choose to upgrade first those bridges that would cost them more not to upgrade. Because funds are limited and capacity demands are growing, railroads need to develop MRR strategies that enable safe and cost-effective operations for the increasing demands of the future.

Railroads' revenue is based on moving freight through their network; however, bridge deficiencies often require speed restrictions to ensure safe operations. For example, the territory of one Class I railroad is divided into subdivisions, which are portions within the territory. Subdivisions have maximum speeds for both passenger and freight traffic based on both safety and infrastructure sustainability. A time table is a document issued by the railroad for each subdivision. Based on infrastructure conditions, time tables list specific portions of the subdivision (between mile posts) assigned to lower maximum allowable speeds than those of the subdivision. Railroad managers invest MRR every year to guarantee the safety of infrastructure to allow the operations of traffic at the speeds listed on the time table. A General Bulleting Order (GBO) is updated every day by the railroad and lists the updated additional speed restrictions for specific portions of their territory. If unsafe bridge responses at a given speed are identified, a temporary (or permanent) slow order is issued and reflected in the GBO. Slow orders cause cost and stress to railroad capacities and operations.

1.4 Bridge management programs

Even with more than 40% of the nation's freight tonnage being carried by railroads, the FRA has traditionally done fairly little monitoring of the condition of railroad bridges (Miller, 2007). This situation is attributed, in part, to the FRA accepting that railroad bridges are safe (Richards, 2007a), even when the current network was built over 100 years ago. According to the U.S. Government Accountability Office (GAO), there has not been a fatality associated with a rail bridge failure since 1957 (Miller, 2007). The FRA trusts the extensive maintenance work performed annually by railroads keeping bridges safe and reliable until they are replaced (Richards, 2007b).

On September 13, 2010, the FRA implemented new regulations regarding railroad bridge management (FRA, 2010a) that became mandatory for all railroads in 2012, requiring to include bridge management programs for all of their bridges. Until 2012, the Federal Government allowed railroads to conduct their own inspection, maintenance, rating, and safety programs. In 2007, attention towards safety of railroad bridges notably increased after the collapse of a small railroad timber trestle that was carrying elements of the space shuttle (Richards, 2007a). This accident added to the overall concern of the general public about bridges, both highway and railroad after the collapse of the I-35W Mississippi River highway bridge on August 1, 2007 (Reid, 2008). Under the new regulation, all railroad bridges need to be structurally inspected and rated at least annually.

One of the key problems to determine optimal policies for MRR decisions is the ability to determine the condition of the bridge from regular inspections. Bridge malfunctions may not always be captured by regular visual inspections and can eventually evolve into an unsafe bridge condition and eventually into unsafe rail operations. In this context, bridge response to revenue service traffic is believed to be a proxy for bridge health (i.e., if the bridge is not moving while the trains are crossing, then it is assumed to be in good shape). Indeed, a top research priority of the railroad bridge structural engineering community in North America is determining bridge displacements under revenue service traffic (Moreu and LaFave, 2012). Moreu, et al. (2014) provided preliminary results relating bridge displacements to service condition; however, measuring bridges responses under traffic is complex, sometimes not possible, and currently limited to subjective observations. As a result, when bridge response data is employed in MRR decisions, it is typically only qualitative in nature. If an objective relation for the serviceability of a given bridge could be established, railroad bridge managers could prioritize their MRR decisions using objective information of each bridge.

1.5 Railroad bridge networks costs assessment

The cost to the railroad to maintain the bridge network is comprised of two components: (i) the operational costs (OC) and (ii) the costs associated with MRR decision for the bridge network. Operational Costs (OC), as defined herein, have two components. The first is the Operational Expense (OE), or the expense beyond MRR investments to maintain the bridge network to meet operational needs. The second component is the Lost Revenue (LR) to the railroad associated with not doing MRR on specific bridges. Railroads decrease the speed of trains over bridges of poor condition, assuming the associated expenses related to traffic delay. The Total Network Cost (TNC) is the cost of MRR, plus the OC, which is uncertain. Thus, the goal is to choose MRR policies that will minimize the expected value of the TNC. Figure 1.3 shows the relationship between MRR investments and the TNC associated with maintaining the bridge network over a specified period of time. Low investments of MRR are associated with high expenses (OE), i.e., if the bridge network is poorly maintained the cost to operate it will be higher, whereas large MRR investments increase the bridge condition and reduce the expenses associated to poor bridge performance. For example, in 2009 President Obama justified funding the replacement of four railroad bridges considering the hypothetical expenses of interruptions to both marine and rail operations and applying the Truman-Hobbs Act (United States Coast Guard, 2009; United States Government Publishing Office, 2009). While the MRR costs are deterministic, the OC are uncertain, therefore current MRR decisions try to conservatively minimize OC. An optimal MRR policy would minimizes the total costs to the network (TNC).

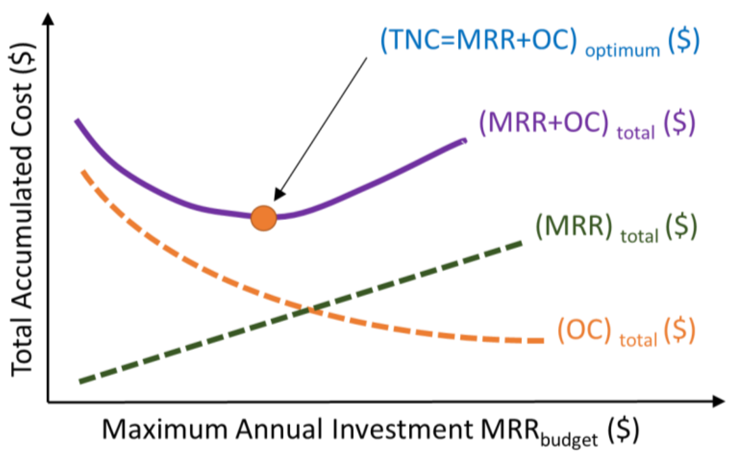


Figure 1.3. Minimization of Total Network Costs (TNC).

1.6 SHM of railroad bridges using WSS

The 2025 Vision from the American Society of Civil Engineers (ASCE) indicates that “relying on and leveraging real-time access to living databases, sensors, diagnostic tools, and other advanced technologies to ensure informed decisions are made” (ASCE, 2013a) (see Figure 1.4). The report card from the ASCE to railroads has a grade of C+, higher than the America’s infrastructure grade overall grade of D+ (ASCE, 2013b). In the last two decades, Structural Health Monitoring (SHM) of civil engineering infrastructure has quickly evolved from the research arena to full-scale, real field applications, opening new avenues of research with practical applications attractive for infrastructure owners. By installing sensors and collecting data from points of interest in a given structure at the field, the structural responses can provide valuable information to owners in real-time about that structure. Similar to the monitoring of the health of one patient, these measurements can be used by owners (or managers) of bridges as indicators of their structural health. In the last five years, the use of wireless smart sensors (WSS) has brought more effective ways of data collection for infrastructure owners, since they can be easily and quickly installed at the field, and successfully collect bridge responses (Spencer et al. 2011).

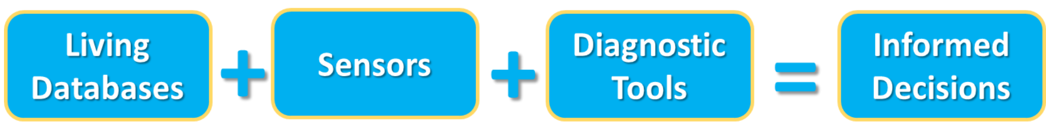


Figure 1.4. ASCE 2025 vision.

1.7 Dissertation overview

This dissertation proposes an initial framework for the consequence-based management of railroad bridges for making network-wide MRR decisions. Because the operational costs are uncertain, the goal established here is to minimize the expected value of the total network cost. Critical to the framework is the ability to assess bridge service condition. The proposed framework employs fragility curves to this end, which relate service condition limit-states to bridge displacement under revenue service traffic. The operational costs associated with these service conditions can be used to estimate the total costs of a given MRR policy. In this way, MRR decisions can be prioritized, minimizing the total network costs to railroad operations. Additionally, measured bridge data can be used to update periodically the fragilities to have more accurate estimates of the bridge condition. This framework provides a consistent approach for intelligent management of railroad bridges, and more specifically, for the prioritization of railroad bridge MRR decisions. Using this framework the rail owner can identify the most efficient use of a limited budget while maintaining safe railroad operations. An example illustrates the development of each component for the framework. The final objective of this research is to provide a new tool to better inform MRR decisions of existing railroad bridges networks.

A short description of each of the chapters of this dissertation is provided below:

Chapter 2 lays the foundation for the work developed in this dissertation by reviewing literature related to the topic. This literature covers concerns related to railroad bridge safety, management, and monitoring; as well as novel work involving wireless smart monitoring advances and efforts. This information will allow the contributions of this dissertation to be put in their proper technical perspective.

Chapter 3 will describe the results of a survey-based study about railroad bridges and structural engineering research topics. This survey-based study identifies collecting railroad bridge displacements under revenue service traffic as a top research need of the rail industry.

In Chapter 4, a new methodology to prioritize MRR decisions will be developed to assist the main concerns of railroad bridge managers. This framework will assess railroad bridge service conditions for management decisions using bridge responses under revenue service traffic using fragility curves. The proposed framework uses: (i) transverse bridge displacements easily measured at the field and (ii) railroad bridges service condition limit states. This framework helps prioritize railroad bridge maintenance, replacement, and repair (MRR) decisions. Railroad managers can use the displacement collected at the field from bridges of unknown condition and obtain the different probabilities of impacting railroad operations. Using this framework the rail owner can maximize the safety of railroad operations, using bridge measurements to quantify their effects into rail operations and levels of service (LOS).

Chapter 5 will include results from bridge monitoring efforts in the field proving that displacements can be used as bridge condition limit states, as a function of train loads and speeds. Focus will be placed on timber trestle bridges, which comprise approximately 24% of the total inventory length of railroad bridges in the U.S.

Chapter 6 will provide results for reference-free displacement estimations using wireless smart sensors for a timber railroad bridge. Reference-free accelerations collected with Wireless Smart Sensors (WSS) will be used to estimate railroad bridge displacements under live train loads. The results will show that transverse displacements of timber railroad bridges can be estimated using WSS, and that WSS can be an effective tool for both campaign and remote monitoring of railroad bridges (with applications for bridge assessment).

Chapter 7 will illustrate how this new framework can become a tool to quantify the most efficient use of a fixed budget by optimizing MMR decisions of bridges based on their impact to current (or future) railroad operations for a given network.

Chapter 8 will describe the use of WSS towards the development of this framework. To validate the ability of WSS to be used to assist railroads, one field application shows the potential of WSS to be used under revenue service traffic. Informed decision will include synchronized wireless weight-in-motion; magnetic strain response measurements for both campaign and autonomous monitoring; and fatigue prediction applications.

In Chapter 9, a summary of the contributions for industry and research found in this dissertation will be presented, major conclusions of this work will be outlined, and specific directions for future work on these areas will be discussed.

CHAPTER 2 LITERATURE REVIEW

The literature review related to the work presented in this dissertation is divided in three main groups. The first group covers the current status of safe, efficient railroad bridge engineering management in North America. The second group reviews current SHM methods, WSS systems and features, and specific applications available in use up to the date related to bridge safety and management. The final part of the literature review provides background information of fragility curves and their use for making decisions at the network level. Additionally, current studies exploring new applications of using sensors to collect data that can assist owner to make decisions about the management of their structures are examined.

2.1 Railroad bridges in US

2.1.1 Railroad bridges differences with highway bridges

In many cases, bridge engineers overlook the differences between railroad bridges and highway bridges, trying to apply research methods conceived for highways to railroads. In fact, nowadays highway bridges are typically more commonly studied by the structural and transportation engineering community. However, railroad bridges pose unique characteristics caused by the environment in which they function and that affect their design, construction, maintenance and management. Consequently, based on these essential differences between highway and railroad bridges, current challenges in railroad bridge management need to be addressed specifically within the railroad environment.

Sorgenfrei and Marianos (2000) pointed out seven main differences between railroad and highway bridges:

1. Live load to dead load ratios are much higher in railroad bridges.
2. Impact factors are higher in railroad bridges.

3. Railroad bridges tend to prefer simple span structures versus continuous structures that will allow them for quick replacement in emergency situations.
4. Interruptions to service are always to be kept to a minimum in the railroad industry, and hence constructability and maintenance are always planned and executed without traffic interruptions.
5. Since the bridge structure supports the railroad track carrying trains, the interaction between track and bridge movement and behavior should be considered as this affects trains on the track.
6. Seismic performance on railroad bridges controls their design differently than for highway bridges. In the past, railroad bridges have behaved well under earthquakes.
7. Railroad bridges are typically expected to last longer than highway bridges.

2.1.2 Railroad bridges design and capacity

In the past, American railroads did not design their bridges following any code, and it is generally accepted that the first national regulation appeared only in 1905 (Unsworth, 2010). Railroad managers make maintenance decisions using information from bridge inspections, including observations of bridge responses under revenue service traffic. Railroads design and rate railroad bridges using a standard sequence of loads called Cooper E-load, which increasing load following the index E (AREMA, 2014). According to Unsworth, the weight/car has augmented rapidly in the last decades and the capacities are being exceeded in old bridges (Figure 2.1).

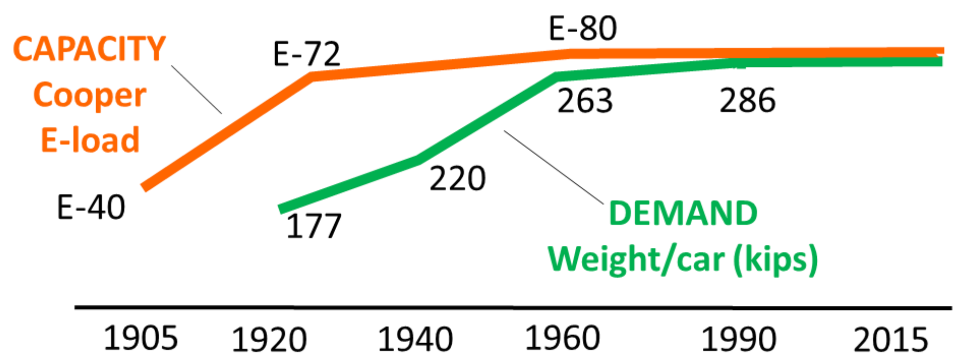


Figure 2.1. Railroad bridge capacity and weight/car demand comparison.

2.1.3 Railroad bridges classification

When figures from both railroad and highway bridge networks are compared, there are substantial differences. The Structural Engineering Institute of the American Society of Civil Engineers (ASCE/SEI) inventoried of the number of highway bridges in USA (2008) grouping them in steel, concrete and timber. The FRA also has put together inventories of their bridges using the same grouping approach under the Railroad Safety Advisory Committee (RSAC) (FRA 2008a, 2008b) (Figure 2.2, Table 2.1). Particularly, steel highway bridges represent only 31% of the total, whereas timber bridges are 5%. In the other side, concrete (41%) and prestressed concrete (22%) add up to 63%. These percentages differ substantially from the railroad bridge population. In the past, railroad bridges have been classified in the past in numerous occasions towards bridge performance. Literature reviews included past and current classifications carried in related bridge manuals (AREMA, 2010) and surveys conducted by the FRA in both 1993 and 2008 (2008b) (Figure 2.2). Concrete bridges include masonry, and steel bridges include iron. Other past studies included past publications by ENSCO (1994), Committee 10 Structures Maintenance & Construction of AREMA (2008), and the International Heavy Haul Association (IHHA, 2009).

However, there is not a current reference classifying railroad bridges based on their performance under revenue traffic or their significance to railroad bridge operations.

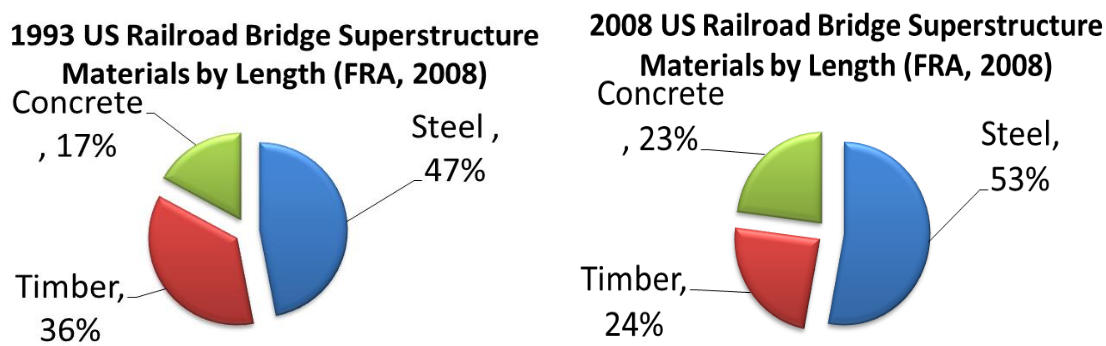


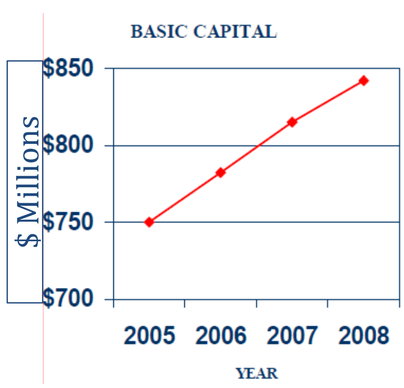
Figure 2.2 FRA Railroad bridge inventories (FRA, 2008a).

Table 2.1 RSCA bridge working group 2008 bridge count for US railroad (FRA, 2008b).

Railroad Classification	Number of Bridges	Miles of Bridges			
		Steel (referred as Metal)	Concrete (referred as Masonry	Timber	Total
Class 1 Freight	60,688	792.26	368.92	278.02	1,439.20
Passenger	2,129	36.16	17.74	0.24	54.14
Short Line & Regional	14,033	106.64	20.24	140.01	266.88
GRAND TOTAL	76,850	935.05	406.90	418.27	1,760.22
1993 Percent		47%	17%	36%	
2008 Percent		53%	23%	24%	

2.1.4 Railroads bridges MRR costs

Railroads manage their bridges inventory using information about their responses under trains. Figure 2.3 shows the cost of bridges to a Class I railroad’s basic capital investment, almost 10% of the total annual basic capital investment for track and property budget (Ferryman, 2008). The percentage remains relatively unchanged from similar data presented three years earlier (Ferryman 2005). The capital invested toward railroad bridges and structures by the railroads, relative to their operating expenses, can be a parameter used to illustrate the importance of bridges to this industry in relationship to their entire capital investments. According to the AAR (2002, 2006, and 2009), the expense (costs) directed towards structures and maintenance of way (i.e., bridges, tunnels, and clearance of track) represent about 17% of their total expenses. Because railroads are private companies searching for possible reductions of in-house costs to increase income benefits, U.S. railroads have promoted and developed studies directed at the cost-effectiveness of retrofitting railroad bridges (Day and Barkan 2003; Resor et al. 2001). However, to date there are no published studies that relates bridge performance under regular operating conditions to bridge management.



2008

Rail	$247.7M
OTM	$81.9M
Ties	$227.9M
Ballast	$67.3M
Bridges	$84.6M
Other	$81.4M
Work Equip.	$19.9M
S & C	$31.7M
TOTAL:	$842.3M

Figure 2.3. 2008 CN basic capital investing (Ferryman, 2008).

2.2 Bridge networks management

Several researchers have developed methods to incorporate quantitative information when developing optimal policies for MRR decisions for highway bridges (e.g., Ravirala et al. 1996; Frangopol et al. 2000). Probability-based highway bridge network studies developed in recent decades have addressed overall performance evaluations based on network connectivity, user satisfaction, and structural reliability (Liu and Frangopol 2004, 2006a, 2006b). Furthermore, probabilistic and reliability optimization methods include simulation-based studies of critical systems, providing cost-benefit analysis tools to assist informed decision-making (Na and Shinozuka, 2009; Ray-Chaudhuri and Shinozuka, 2010; Cremona et al. 2013). However, because (a) highway traffic loads and demands are much lighter and less critical to their bridges than train loads; (b) structural failures are targeted as opposed to serviceability of operations; (c) a linear decay of structural properties over long periods of time (decades) is assumed, and (d) railroad infrastructure management prioritizes the current state (or immediate future), these methods are not applicable directly to management of railroad bridges. Consequence-based information about MRR decisions can improve the management of railroad bridge networks.

2.2.1 Railroad bridge management program

Of significant importance to the railroad industry is keeping the railroad network fluid and free of service disruptions, which can wreak havoc on railroad network operations as well as on the operations of the railroad's customers (New York Times, 2006). Therefore, railroad companies look very closely at the integrity of their railroad bridges. Safety and economics must govern bridge maintenance (Waddell, 1921). According to the IHHA, "the extension of asset life through research and rational assessment is critical to the continued safety and economics of Heavy Haul

(HH) operations” (IHHA, 2009). It can be concluded that the efficient usage and management of bridges guarantees safe and profitable operations for the railroads and their customers (Figure 2.4 shows the U.S. railroad network). Existing literature about bridge management compiles policies and advances directed in general to highway bridges, but not specifically to railroad bridges (Ryall, 2001; Yanev, 2007). As a consequence, AREMA describes the requirements of a Bridge Management System (BMS) for each railroad bridge in the new Chapter 10: Structures, Maintenance, and Construction (AREMA, 2014). However, the development of specific detailed methods and policies is still lacking since this is a new requirement of the railroad industry.

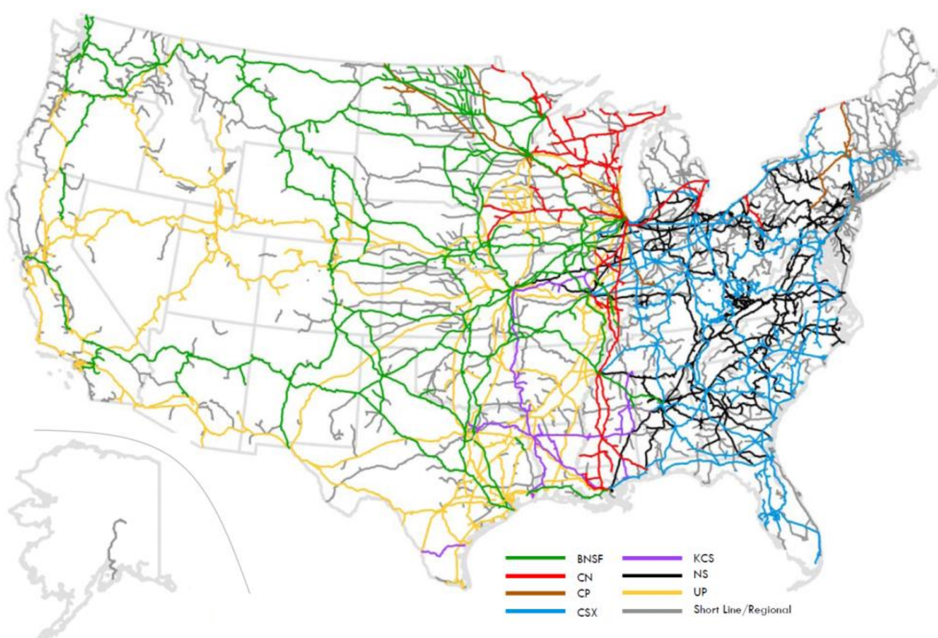


Figure 2.4 The U.S. railroad network (AAR, 2007).

2.2.2 Federal Railroad Association (FRA) railroad bridge safety 2010 regulation

In July 15, 2010, the FRA published a final rule on Bridge Safety Standards, establishing new federal safety requirements for railroad bridges. In September 13, 2010 (90 days later), the new rule became effective. The new regulation can be found under reference 49 of the Code of Federal

Regulation (CFR), Parts 213 and 237. In general, the contents of this new rule focus on determining both the contents and obligations related to railroad bridge management programs in the U.S. In more specific terms, this new regulation enforces new instructions pertaining to inspections, load capacity determinations, repairs, and modifications for railroad bridges (FRA, 2010a).

49 CFR Part 237, Subpart B – Railroad Bridge Safety Assurance, "Adoption of bridge management programs," points out that "… every track owner shall adopt a bridge safety management program to prevent the deterioration of railroad bridges" (p. 41303, section 237.31). This same section enforces the adoption of a bridge management program for each Class I railroad by March 14, 2011, and before that date for other different railroads carriers, but not later in any case than September 13, 2012 (FRA, 2010a).

Additionally, "Content of bridge management programs" (section 237.33) defines the requirements for all bridge management programs. According to this section, to comply with the new regulation the contents of the bridge management program should include:

1) an accurate and detailed inventory of their railroad bridges,
2) a record of the safe record capacity of each of the bridges,
3) a provision to obtain and maintain the design documents of each bridge if available, and to document all repairs, modifications, and inspections of each bridge, and
4) a bridge inspection program. The specific requirements of the bridge inspection program required by the new regulation can be found in the reference section under FRA (2010a).

2.3 Railroad bridges inspections

Railroads conduct different kind of scheduled inspections as part of their bridge management policy to ensure that their capital investment is cost-effective, productive, and safe. After the 2007 collapse of a short line railroad timber trestle bridge in Alabama carrying space shuttle elements, attention toward railroad bridge inspections increased dramatically (Richards, 2007a).

Railroad bridge inspections are critical to railroad bridge management in North America. The *AREMA Bridge Inspection Handbook* (AREMA, 2008) states that railroad bridge inspections directly affect the actual operations for the entire network. If a particular bridge inspection finds unsafe conditions for one particular bridge, railroad traffic could be interrupted. If the bridge inspector determined that the findings from their inspection compromise the safety of trains running over it, they could immediately request a slow order for that particular bridge, or even completely divert/stop the traffic expected for that particular bridge.

Various studies about the different types of inspections in the railroad industry concluded the following:

1. The FRA provides statements about the safety of railroad bridges (2000, 2005), but even when FRA is aware of the current inspection methods and procedures of the railroads, the FRA does not take responsibility for them (Davids, 2010).
2. According to Kube (2007) there are many other aspects of the current bridge inspection procedures in the short and medium railroads that could be investigated.
3. As presented by Sweeny and Unsworth (2008, 2010) (Figure 2.5) and Lozano and Kavars (2009), there are many cases in which railroad bridges are not accessible to inspectors except by rail.

In summary, there is an interest from railroads in developing inspection standards that can be followed by railroad inspectors but these references to date do not allow for quantitative measurements at the field. Monitoring is not included in current bridge inspection standards in the railroad industry.

Figure 2.5 Railroad bridge inspection conducted from the railroad track (Sweeney and Unsworth, 2008)

2.4 Railroad bridges forced vibration

Railroad managers want to control and observe the response of railroad bridges under revenue service, because they are worried about the effect of the heavy load of the train crossing the bridge (Figure 2.6). In fact, the response of the bridge under the train is a complicated 3D non-linear dynamic problem that involves the interaction of the bridge, the track structure, and the train. Railroad trains can weigh as much as the bridge for given bridge configurations (new steel bridges) or even larger (timber bridges).

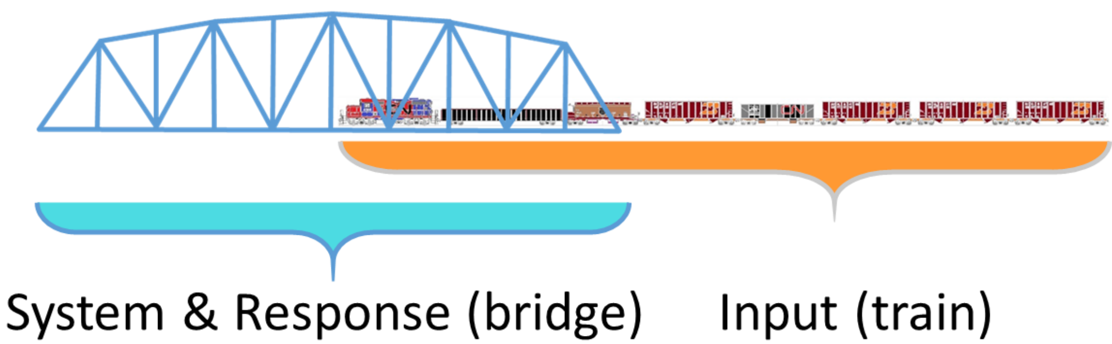


Figure 2.6. Challenges of forced vibration of railroad bridges under revenue service traffic.

If the problem of the bridge responses under dynamic loading is assumed to be modeled with the equation of motion of a single DOF system, and the input force is undetermined:

$$m\ddot{u} + \zeta\dot{u} + ku = f(t) \tag{2.1}$$

We can rewrite my EOM in the state-space representation:

$$\dot{Z} = AZ + BU$$
$$Y = CZ + DU \tag{2.2}$$

where

$$\dot{Z} = \begin{bmatrix} \dot{z}_1 \\ \dot{z}_2 \end{bmatrix} = \begin{bmatrix} \dot{u} \\ \ddot{u} \end{bmatrix} = \begin{bmatrix} \underset{\sim}{0} & \underset{\sim}{I} \\ -M^{-1}K & -M^{-1}C \end{bmatrix} \begin{Bmatrix} u \\ \dot{u} \end{Bmatrix} + \begin{bmatrix} \underset{\sim}{0} \\ M^{-1} \end{bmatrix} \{ f(t) \} \tag{2.3}$$

and

$$Y = \begin{bmatrix} u \\ z_2 \\ \dot{z}_2 \end{bmatrix} = \begin{bmatrix} u \\ \dot{u} \\ \ddot{u} \end{bmatrix} = \begin{bmatrix} \underset{\sim}{I} & \underset{\sim}{0} \\ \underset{\sim}{0} & \underset{\sim}{I} \\ -M^{-1}K & -M^{-1}C \end{bmatrix} \begin{Bmatrix} u \\ \dot{u} \end{Bmatrix} + \begin{bmatrix} \underset{\sim}{0} \\ \underset{\sim}{0} \\ M^{-1} \end{bmatrix} \{ f(t) \} \tag{2.4}$$

For any given undetermined loading condition, the response of this system can be represented by the transfer function of the response to the forced vibration. For different loading scenarios there will be resonance problems that are of changing nature under different loading scenarios (Figure 2.7). In general, railroads want to reduce large responses of railroad bridges under moving loads by controlling the speeds of the traffic crossing the bridge. For a given bridge that shows poor train operations, the first decision to be made by railroad management is to reduce the maximum allowable speed of traffic crossing the bridge.

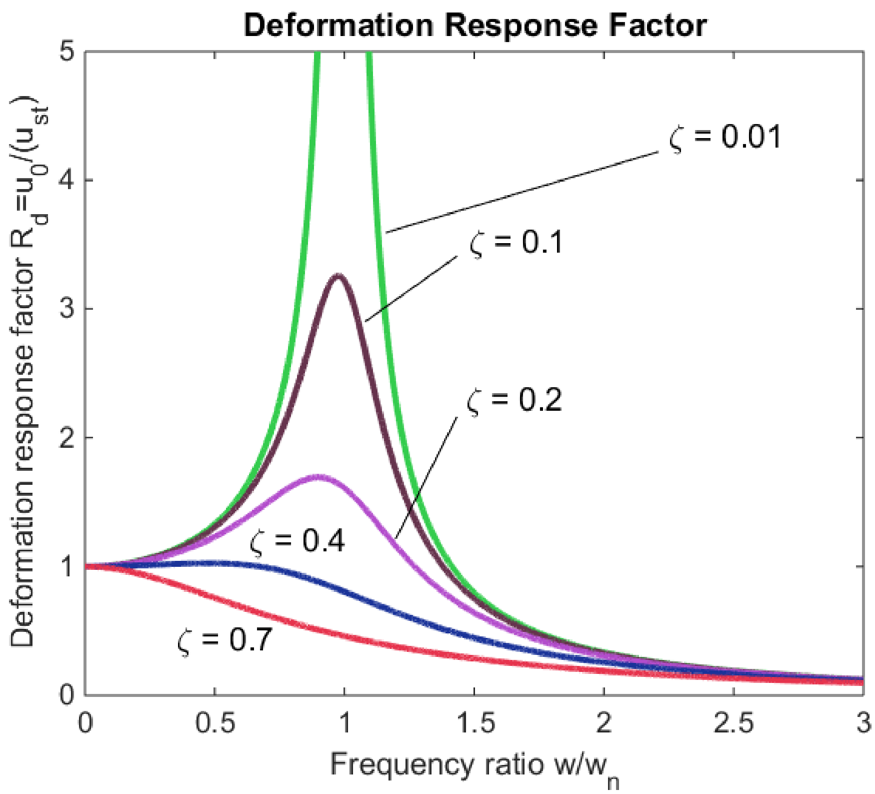


Figure 2.7. Railroad bridge forced vibration under undetermined input force.

2.5 Railroads bridge management based on rail operations

Railroads bridge engineering departments need to ensure that their infrastructure guarantees safe operations to increase the profit of their stakeholders (Hay, 1982): interruption to traffic must be avoided at all terms. FRA (2014) divides track in five classes corresponding to the maximum allowable operating speed. Maintaining adequate track capacity to address expanding passenger and freight needs is one of the largest challenges in creating a financially competitive rail network. Operating speed on a given track affects its capacity, which is the ability of a given railroad to move a given volume of traffic over a specific line under given Level of Service (LOS) (Lai and Barkan, 2009). Temporary Slow Orders (TSO) affect track capacity, and when caused by bridge condition, can evolve in bridge repairs or replacements (Moreu and LaFave, 2012). Reducing the track class because of unplanned bridge work affects negatively railroad operations and reduces financial profit. Risk of infrastructure malfunctioning and affecting railroad operations (i.e. with emergency slow orders) needs to be reduced.

Railroads control and measure track geometry within safe tolerances to ensure the safety of their operations, but there are no metrics limiting bridge performance under traffic. Track departments conduct track geometry inspections regularly collecting information about the track misalignments in vertical direction, transverse direction (Table 2.2), and gage separation (Table 2.2). If the measurements exceed thresholds, the FRA requires that the railroad immediately remedies the situation and reduce operating speeds to levels of safety until it is solved. If the track information is collected near a bridge and provides warning about track defects, the track department contact bridges and structures. However, bridge malfunctioning may not always be captured by regular track performance and go unnoticed, and can eventually evolve in unsafe bridge condition and eventually in unsafe rail operations (Moreu, 2014). Railroads must ensure bridges safety by conducting annual inspections and maintaining bridge management programs (FRA, 2010a). There are recommendations about controlling bridge displacements under traffic, but there are no limits about which quantities are the maximum tolerable under train traffic to ensure safety of traffic is not compromised.

Table 2.2. FRA track alignment limits (2015).

Track	**Tangent Track Alignment Limit** The deviation of the mid-offset from a **62-foot line** may not be more than—(inches)
Class 1 track	5
Class 2 track	3
Class 3 track	1 ¾
Class 4 track	1 ½
Class 5 track	¾

Table 2.3. FRA track gage limits (2015).

Track	The gage must be at least	But no more than
Excepted track	N/A	4’10 1/4”
Class 1 track	4’8”	4’10”
Class 2 and 3 track	4’8”	4’9 3/4”
Class 4 and 5 track	4’8”	4’9 1/2”

Railroads classify their route lines in two general categories. The main line carries larger amount of traffic at higher speed, and it is in general composed of track classes 3, 4, and 5. Side lines or branch lines carry smaller amount of trains/day and at lower speeds, with track classes 1, 2, and 3. Figure 2.8 shows a hypothetical branch and main line with two different slow orders in two different bridges. In general, the railroad transportation department will try to prioritize budget to repair or replace the bridge in Subdivision 5 with a higher priority than the bridge in Subdivision 3 because Subdivision 5 is on the main line. Consequently, the slow order consequences are different depending on the track class of the subdivision where the bridge is.

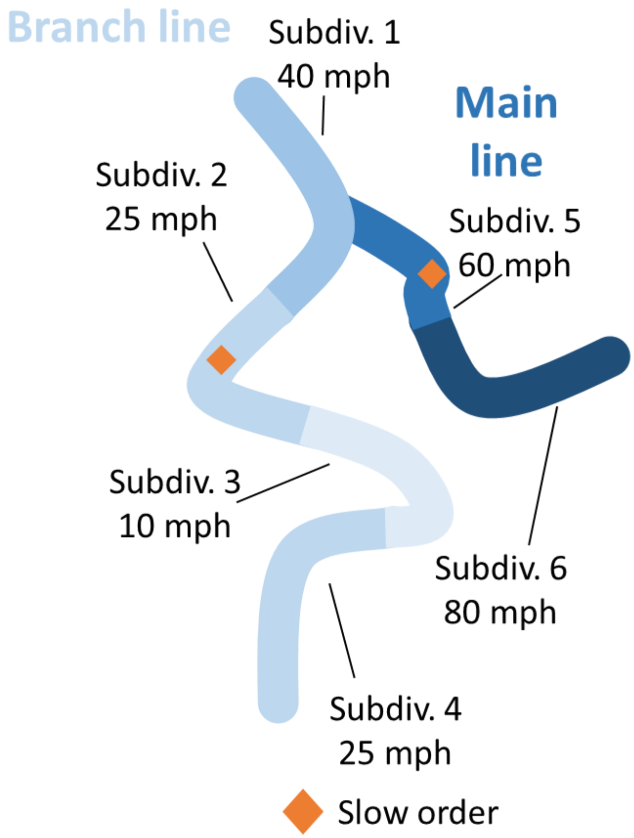


Figure 2.8. Example of a branch line and main line traffic and bridge slow order.

2.6 Railroad bridges monitoring

In the past, American railroads studied the implementation of sensors to prevent railroad bridge catastrophes, but found that the installation of sensors to manage safety of railroad bridges was too expensive and would not be justified, due to the large initial investment that would need to be made.

In 1981, after a bridge-related derailment over a damaged bridge in 1979 in Devils Slide, Utah, the FRA estimated the cost for detection devices in the U.S. railroad bridge network at $850 million to install and $85 million a year to maintain. Subsequent train derailments – like the one in Secaucus, New Jersey, in 1996, preceded by the 1993 Alabama derailment, which killed 47 people and injured 103 (see Figure 2.9) – stirred the debate about sensor systems installation. Even discussions following these accidents found the concept appeal and interesting for research and study, however, in the words of an Amtrak spokesperson “we’ll have to see if it could be effective, reliable and truly useful” (Applebome, 1993). The Amtrak accident at Alabama (Gendisasters.com, 2014) occurred right after a freight train crossed the same bridge shortly before and reported no abnormal riding conditions over the bridge. The consequence of the accident was that railroads in North America re-started considering using sensors for long-term monitoring of railroad bridges to increase safety of rail operations by remote monitoring of bridges.

Figure 2.9. Alabama bridge accident, 1993 (Gendisasters.com, 2014)

In 1994, the FRA sponsored a study towards monitoring technologies and methods for the structural health monitoring (SHM) of railroad bridges in US. The conclusions proved that the installation of monitoring integrity in the entire population of railroad bridges in the US would cost more than the money lost by railroad bridge related accidents for a 25 year period of study, even when the false alarm costs to the railroads were included in the estimates (ENSCO, 1994). The cost of installing sensors in bridges was estimated from a few thousand dollars per bridge to as much as $40,000. The total estimate for installing sensors along the entire railroad bridge network reached billions of dollars, with an estimated cost of $60 million a year for operation and maintenance (Perez-Pena, 1996). Additional information and research investigating the cost-effectiveness of railroad bridges monitoring systems can be found in the study "Overview of Railroad Bridges and Assessment Methods to Monitor Railroad Bridge Integrity" (ENSCO, 1994).

Railroads are currently interested in adding instrumentation to their bridges that can inform them about their performance under in-service trains. In 1997, Tobias and Foutch pointed out that steel bridges constructed over a hundred years ago need to be monitored to ensure that the loads experienced do not translate into fatigue failures or deficiencies (Tobias and Foutch, 1997; Unsworth, 2003). As identified in studies by Byers and Otter (2006), there is a significant and growing interest from the railroad engineering community to collect data from bridges in the field. Otter et al. (2012) recently published work identifying the needs of bridge monitoring systems based on railroad bridge service interruptions. Researchers used the stress of dead loads in eyebars in steel railroad bridges to prioritize repair work (Mazurek, 2010). DelGrego et al. (2008) published monitoring work measuring the performance of a railroad truss bridge and directing repair work based on those measurements. The recent proliferation and development of more effective, capable, and affordable sensors in the last 10 years identify field instrumentation as a particular area for research by railroad institutions and affiliated laboratories, as indicated in the AREMA President's column (Unsworth, 2011).

Railroad managers seek monitoring systems that can collect loading information during their regular inspections because train loading is the most critical demand for the durability, safety, and efficient management of rail infrastructure (see Figure 2.10). Past studies have explored structural health monitoring (SHM) of railroads using information of the loading into the system (Barke and Chiu, 2005; Karoumi et al. 2005). However, their proposed weigh-in-motion devices are expensive and can be implemented in the field only by railroaders experts in this area or consultants dedicated to this effort. Furthermore, current instrumentation has limited portability for railroad environments: installation time is limited because of train traffic and access to the structure is difficult because the unique separation of railroads from other means of transportation. Banerji and Chikermane (2012) proposed simplified sensing of both rail and structure to match clients' needs and requirements. However, their instrumentation was wired, hence requiring time and substantial efforts for each measurement. Consequently, this approach is not designed for short-term applications (campaign monitoring). Even for the monitoring of only one element within the bridge, the entire deployment would require substantial investment of money, equipment, and personnel. Furthermore, current approaches are not designed to be carried from bridge to bridge. In summary, current monitoring tools to measure train loads and rail infrastructure responses to these loads are costly, complex, and cannot be used for day-to-day management operations in rail environments.

Figure 2.10. Railroad bridge under revenue service traffic.

2.7 Structural Health Monitoring (SHM)

Structural Health Monitoring (SHM) of structures can unfold information about their performance to help determine if, based on intelligently selected structural parameters, structures are performing under “healthy” thresholds. SHM was originally referred as the process of implementing a damage detection strategy for civil engineering infrastructure, aerospace or mechanical engineering systems (Sohn et al. 2003). SHM is today attributed in a broader sense today as assessing the ability of structures to carry loads and structural behavior over time. SHM applications include, but are not limited to: the control construction procedures; the verification of structural properties after extreme events; and checking predetermined invariables or factors throughout the entire expected life of a structure.

Civil engineering structures are typically large, and are built on site conditions and under environmental surroundings that directly affect the properties of the structure once it is finished. Consequently, the structural properties of these complex, large infrastructure systems can’t be properly modeled because of the numerous and unknown variables affecting both their materials and mechanical systems from the very first days of their construction. In any event, when time and money are available, these structural systems ought to be modeled on one by one case, their accuracy being confirmed using data collection from the field.

In the last two decades, civil engineering has developed research of the dynamic properties of structures that can assist structural engineers assessing their structural performance (Doebling et.al. 1996; Chong et al. 2001; Chang et al. 2003; Brownjohn, 2007; Farrar and Worden, 2007). From the early stages of SHM development and research, the large scope in mind has been to propose tools and methods that can assist the proliferation of intelligent infrastructure (Aktan et al. 1998). In particular, early studies by Japanese engineers showed in the 1990s the potential of using vibration measurements for structural capacity assessment (Abe, 1998). Long-span bridges are being extensively monitored today, as presented at the 2008 conference by the International Association for Bridge Maintenance and Safety (IABMAS), entitled "Bridge Maintenance, Safety, Management, Health Monitoring and Informatics" (Koh and Frangopol, 2008), or by other SHM researchers (Pines and Aktan, 2002; Ko and Ni, 2005).

Infrastructure owners would want to measure specific parameters that can help them determine by themselves the health of their structures. Infrastructures owners are seeking to collect intelligent data that captures the structural performance of their bridges, but want to be more active participants on the health assessment (Kijewski-Correa et al. 2012), with liberty to decide sensor location as well as data collection means and methods.

In general, monitoring bridge responses at specific locations can reduce the number of unknowns about the structural system being monitored and provide with quantifiable information about their performance. Some of the current challenges that require further research and study are identifying the number of sensors to be used, and their optimal placement within a structure (Dove et al. 2006). Furthermore, infrastructure owners want to measure the health of their structure by using parameters they are familiar with and can clearly illustrate if their structures are healthy (or not) at any given time. Prioritizing monitoring to become performance indicators can integrate Performance Based Design (PBD) of complex structural systems with SHM techniques. Frangopol (2007) stressed the importance of using SHM techniques to allow management decisions be made from the performance information being provided. However, even when there has been traditionally interest from the railroad about SHM for the monitoring of railroad bridges, only 9.3% of the results of searching “Structural Health Monitoring of Bridges” corresponds to “Structural Health Monitoring of Railroad Bridges” (Google, 2015), see Figure 2.11.

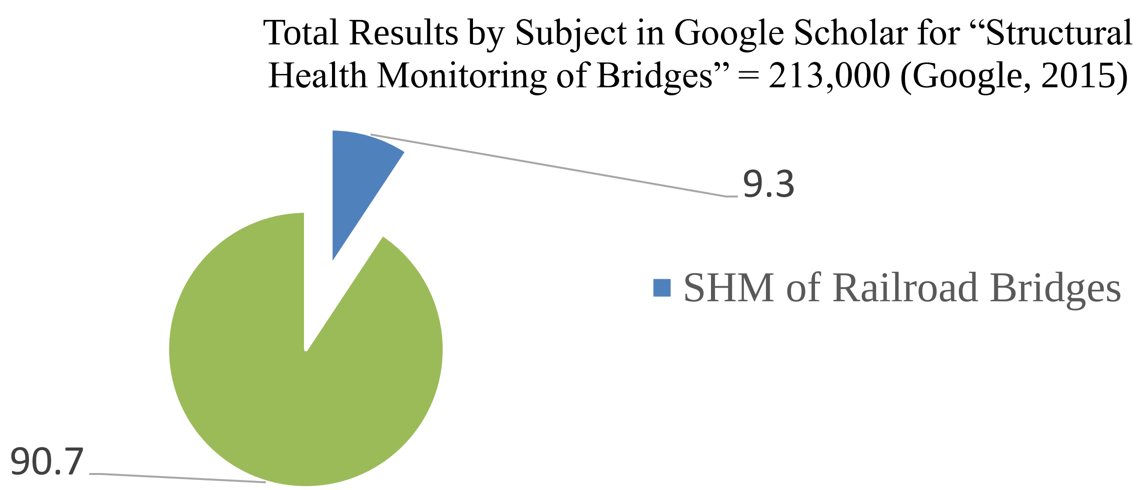


Figure 2.11. Percentage of Google Scholar results of SHM of railroad bridges in comparison to SHM of bridges (Google, 2015).

2.8 Structural Health Monitoring for performance monitoring

There have been numerous efforts in developing SHM for bridge safety, but until today, practical applications that can inform the owner about the current state of performance of the structure are still lacking. The majority of the current SHM approaches collect the input and output to a given structure and use this information to upgrade the structural model. Consequently, what is pursued is to upgrade a model of the bridge being monitored (Figure 2.12). Computer simulations of this model can predict future decay/failure (Figure 2.13).

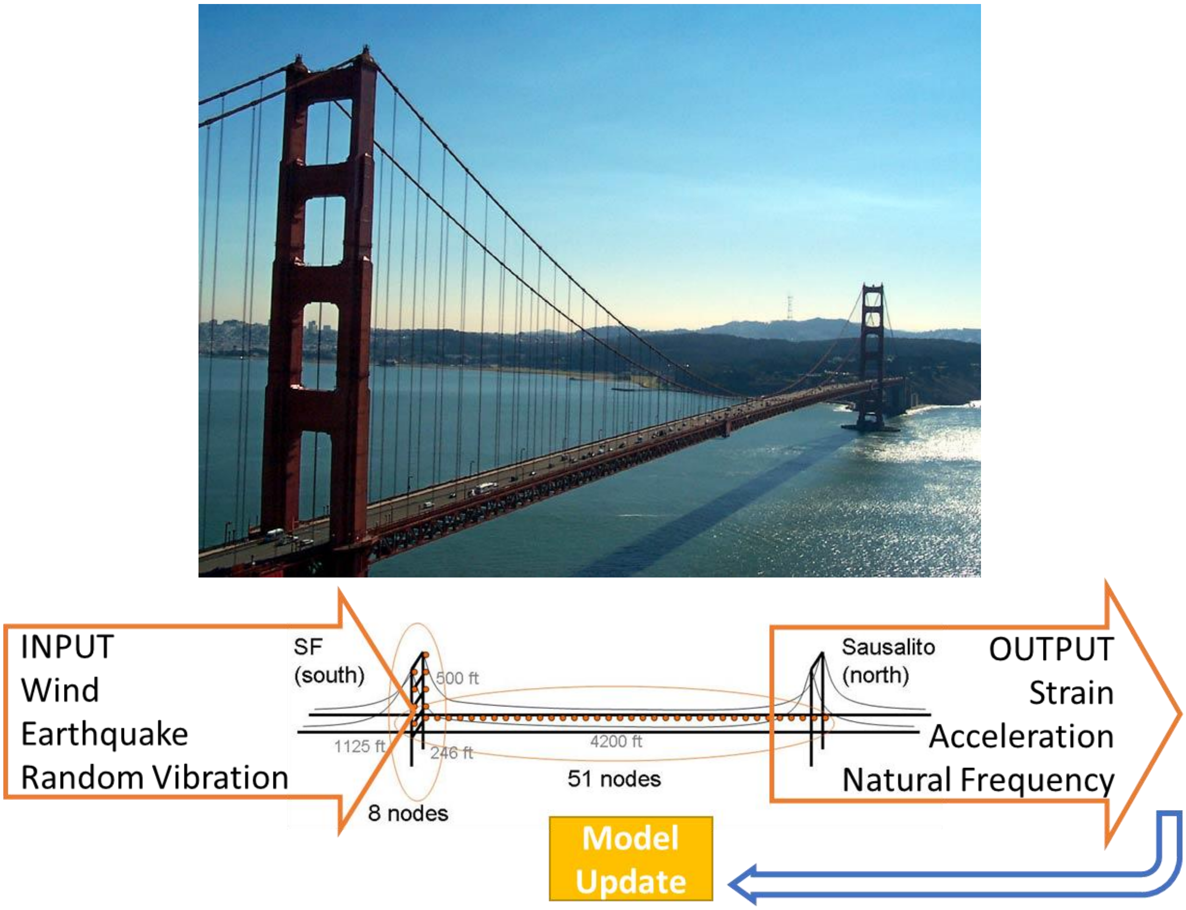


Figure 2.12. Monitoring of the Golden Gate Bridge (Kim et al. 2015).

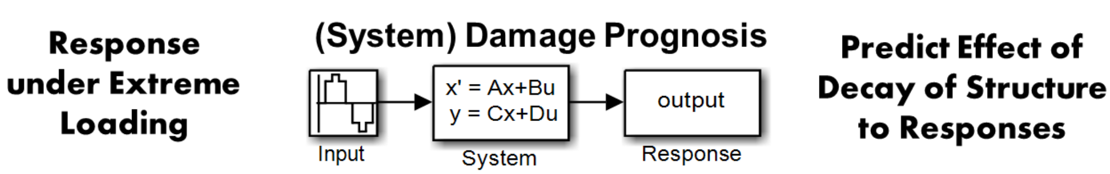


Figure 2.13. Simulation scheme for damage prediction using traditional SHM approaches.

However, the size and complexity of civil engineering structures brings difficulties to the estimation of the changes of structures over time. Additionally, the good models correspond to structures of known conditions, which may not decay in the next 50 years. Consequently, this research proposes a new objective which is developing SHM applications to use information about the responses of the structure to estimate the capacity (Figure 2.14).

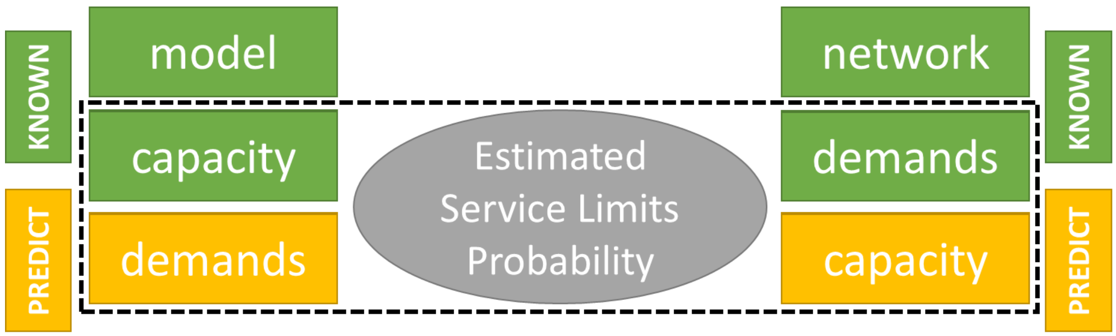


Figure 2.14. Structural Performance Health Monitoring (SPHM).

2.9 Wireless Smart Sensors (WSS)

Traditional structural monitoring systems are comprised of a network of sensors distributed throughout a structure. These networks typically rely on a central source of power and data acquisition and therefore require cables to link the sensors with the power and acquisition hardware hub. Such systems can be prohibitively expensive, often amounting to an installed cost of thousands of dollars per channel. While WSS offer an attractive alternative, much of the associated technology has been available for over a decade; yet limited numbers of practical applications have been found, primarily due to a lack of critical hardware and software elements. To overcome these challenges, the Illinois Structural Health Monitoring Project (ISHMP, 2014) has been developing hardware and software for the continuous and reliable monitoring of civil infrastructure using networks of Imote2-based wireless smart sensors. The open-source software library of customizable services, developed under the ISHMP, implements key middleware services necessary for high-quality sensing, synchronized and reliable network operation, as well as high-level application services, tools, and utilities (Rice and Spencer, 2009). The developed sensor boards for the Imote2 platform provide high-sensitivity acceleration and strain measurements and accommodate signals from other analog/digital sensors (Jo et al. 2010). The Imote2 sensor platform, the Illinois SHM-A board, and the sensor enclosure assembly used for this experiments, are shown in Figure 2.15. The SHM-A board for the Imote2 platform with multi-metric sensing capabilities by Rice et al. (2010) (Figure 2.16) provides temperature, humidity, and light intensity sensing capabilities, in addition to 3-axes acceleration measurements. The temperature sensor on the board was even used for compensating the temperature effects on the acceleration measurements. And the 4th channel of the board is left for integrating external analog sensor having 0~3.3V output. TelosB mote also provides similar sensing capabilities of temperature, humidity, and light intensity.

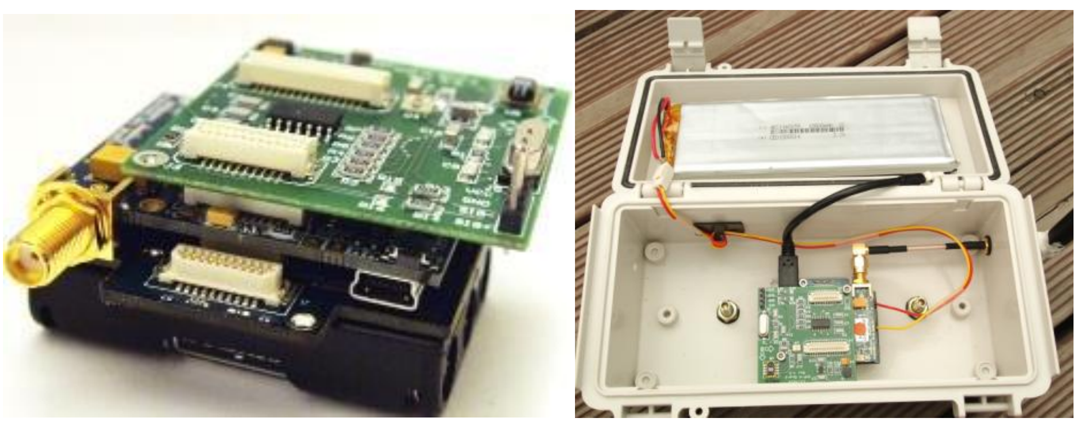


Figure 2.15 (a) ISM400 board stacked on Imote2, and (b) sensor enclosure assembly.

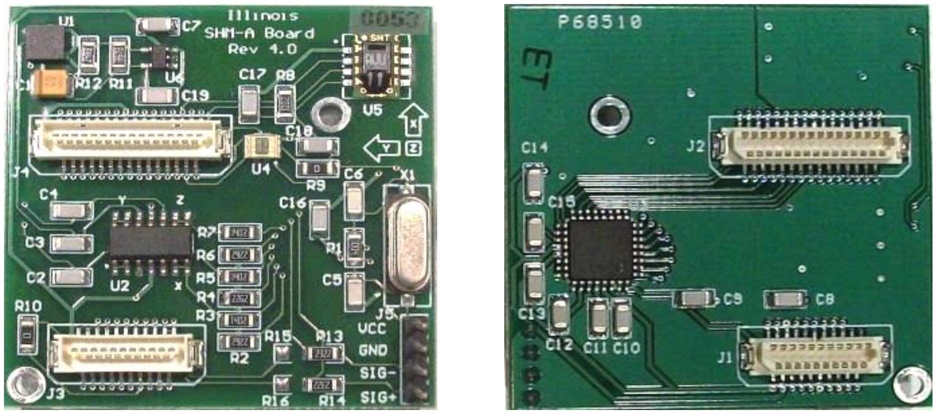


Figure 2.16 Multimetric sensor board (SHM-A board rev.4) for Imote2 platform by Rice et al. (2010); top (left) and bottom (right).

These hardware and software innovations, as demonstrated in the full-scale implementations, form a flexible smart sensor framework for full-scale, autonomous SHM that will be employed for this research (http://sstl.cee.illinois.edu). Under the Illinois Structural Health Monitoring Project (ISHMP), a collaborative effort between researchers in civil engineering and computer science at the University of Illinois at Urbana-Champaign, researchers developed the ISHMP Services Toolsuite. The Toolsuite provides a software framework for continuous and reliable monitoring of civil infrastructure using WSS. This software is available as open source for research purposes at http://shm.cs.uiuc.edu/software.html.

WSS offer an opportunity to provide a portable tool that railroad personnel can quickly and easily install, use, and remove for use on other bridges. Several full-scale deployments have demonstrated the potential wireless sensor technology for monitoring highway bridges. For example, the 2nd Jindo Bridge deployment in Korea consisting of 113 wireless sensors with 669 sensing channels is the world largest full-scale wireless smart sensor networks (Figure 2.17).

Figure 2.17 Jindo Bridge.

2.10 SHM of railroad bridges using WSS

During the last decade, researchers have targeted railroad bridges to propose monitoring strategies for SHM applications using wireless smart sensors, but specific needs by the railroad have not been completely addressed with the past efforts. Wireless smart sensors (WSS) have been currently developed for campaign monitoring applications (Li et al. 2012). Recent studies noted that this application of WSS can assist monitoring highway bridges (Jang et al. 2010, Cho et al. 2010, Spencer et al. 2011), but results about their applicability to the railroad industry is still limited to specific applications. In 2005, researchers used wireless smart sensors to measure timber railroad bridges vibrations under trains, but the measurements could not quantify the structural conditions of the bridge (Moreu and Nagayama 2008). Chebrolu et al. (2008) proposed a new wireless sensor network system for the monitoring of railway bridges that they named “BriMon”; Flammini et al. (2010) proposed implementing WSNs in railroad infrastructure for structural failures as well as hazards and attacks, establishing an theoretical platform for future implementation in the railroad environment; Park et al. (2011a) explored using wireless smart sensors for railroad bridge long-term monitoring applications, and tested their applications using accelerometers and modal analyses; Giles et al. (2011 and 2012) used WSSs to obtain dynamic properties of the unloaded bridge (Figure 2.18). The TRB of the National Academies published a report employing wireless smart sensors for the remote sensing of crack growth on a CN steel railway bridge by Montreal, Canada (Hay et al. 2007); Bischoff et al. (2009) designed and tested an event-based strain monitoring on a railway bridge using wireless technology to measure strain of members under open traffic autonomously. However, these proposals lack at least one of the following: (i) a specific example/s or application/s of the proposed methodology on railroad environments, (ii) portability of the sensing device to make it practical for campaign monitoring, with long time and cost devoted in the installation and removal of the instrumentation, or (iii) ability to collect the input loads by the same monitoring effort. This feature is of special interest for long-term monitoring applications because the owner of the bridge could monitor the change of the responses with the changes of the loads. Specific applications of WSS technology to railroad bridge campaign environment that can simultaneously address the above-mentioned are of interest to the railroad community and are still needed.

Figure 2.18 Government Bridge.

2.11 Timber railroad bridge management

2.11.1 Timber railroad bridges

North American railroads are particularly interested in timber railroad bridges; for some Class I railroads, maintenance and/or replacement of timber trestles currently consumes as much as 40% of their total bridge maintenance budget. The Railroad Bridge Working Group of the Railroad Safety Advisory Committee (RSAC) of the FRA documented 679 km (418 miles) of U.S. timber railroad bridges (also known as timber trestles), comprising 24% of the total inventory length (FRA 2008a). Timber railroad bridges in North America generally consist of a series of 3.7 to 4.6 m (12 to 15 ft) short spans supported on (timber) piles bents (see Figure 2.19.) A significant number of timber spans can be found as approaches to main steel spans. Timber components in these bridges have already exceeded their traditionally accepted life span of 50 years in many locations (Wipf et al. 2000). Not surprisingly, North American railroads have at times had to prioritize maintenance investments toward timber trestles (Uppal and Rizkalla, 1988).

Excessive bridge movements can be a menace to safe rail operations. Caused by transverse rail instability, wheel-hunting movement is a low frequency transverse motion of a railroad car when the wheel flanges contact the rail (AREMA, 2014). Interaction between vehicles, track, and bridge components can increase wheel-hunting movements which can then augment timber railroad bridge transverse displacements. Xia et al. (2008) determined wheel-hunting movements to cause peak amplitudes of transverse deflections for tall railroad bridge piers in China. Researchers have also modeled bridge-vehicle interactions for better assessment of railroad bridge response under railroad traffic (Scheffey, 1964; Tanabe et al. 1987; Frýba 1996, 1999; Yang et al. 2004). Other researchers have emphasized the importance of three-dimensional effects in bridge response (Xia et al. 2006; Wu et al. 2001; Psimoulis and Stiros, 2013; Stiros and Psimoulis, 2012).

Work from Stiros and Moschas (2014) used deflection measurements to find decay of pedestrian timber bridges, through analyzing changes over time of modal frequencies obtained from free attenuating oscillations of transverse deflections. However, studies investigating the relationship between railroad car movements and transverse displacements of timber trestles are not available, in part because such bridges are not easily modeled. Nevertheless, railroads are interested to monitor and study timber trestle displacements and their relation to safe rail operations.

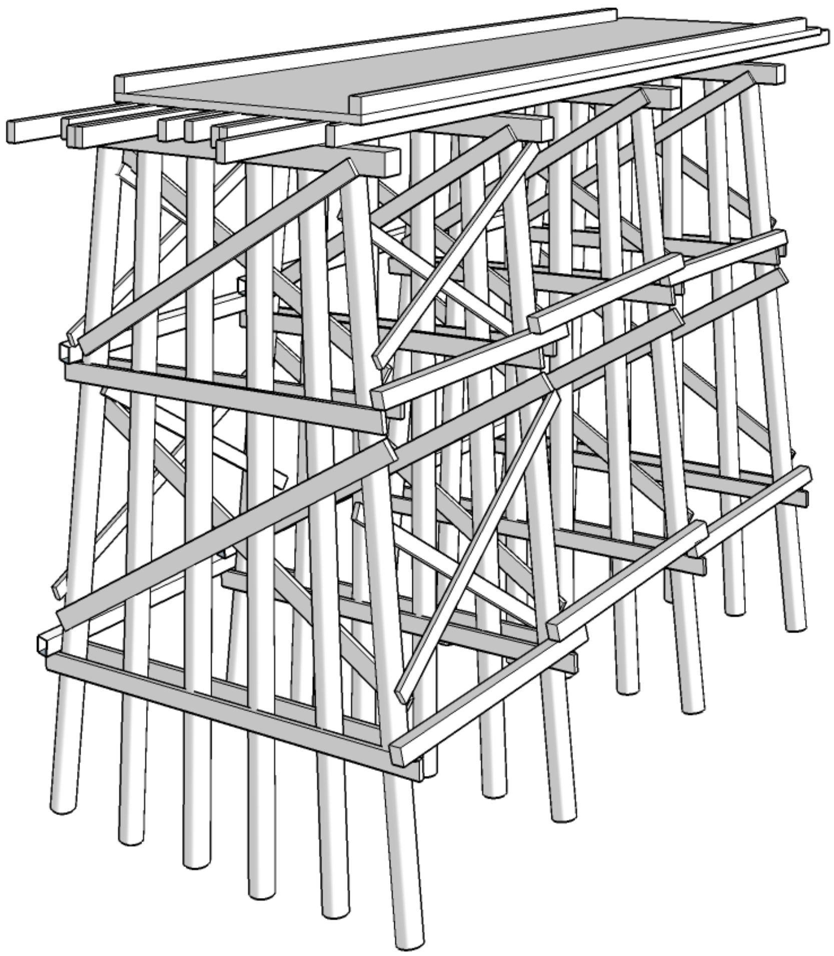


Figure 2.19. Timber railroad bridge (partial view).

2.11.2 Harmonic roll of railroad bridges

Transverse displacements of timber bridges can be further amplified by the interaction of loaded cars running over specific track conditions at moderate speed. This phenomenon is sometimes referred to as “rock and roll”, “harmonic roll off”, or “harmonic roll”. Harmonic roll is an oscillatory motion of heavily loaded railroad cars running on track of low quality that is attributed to high center of gravity of the car, track perturbations, and operating speeds of approximately 24 km/h (15 mph) (Hussain et al. 1980). Harmonic roll is even a train accident cause defined in Appendix C of the FRA Guide for Preparing Accident/Incident Reports (FRA, 2012). Fully loaded railcars typically have a roll frequency response of 0.5-0.8 Hz which is excited when running at given speeds over jointed track with vertical irregularities (Shust and Iler, 2010). Regulations and studies link harmonic roll to various speed ranges: 16-40 km/h (10-25 mph) (FRA, 2005); 19-32 km/h (12-20 mph) (Wolf, 2005); and 21-31 km/h (13-19 mph) (Watco, 2012). Track portions of poor quality can increase the interaction between vehicle and track, augmenting the dynamic component of forces between wheels and rail (FRA, 2005). Because joints are spaced at constant distances, different train speeds increase wheel impacts at different frequencies. Those impact frequencies that match the roll natural frequency of loaded cars cause resonance, which can amplify transverse displacements of bridges under trains.

2.12 Reference-free displacement estimation

Reference-free approaches to estimate bridge displacements have been proposed by several researchers. The collection of displacements in railroad bridges is currently limited to infrequent situations, because of the high mobilization cost associated with installing a reference point by the bridge. A more convenient means to measure bridge displacement is needed. For example, Rice et al. (2011) used low-cost radar-based sensing for the measurement of deflections. For this application, targeted specifically for long-span bridges, a fixed (though remote) reference point is still required. Koo et al. (2013) used a robotic total station (RTS), or theodolite positioning system (TPS), to remotely measure dynamic deflections; this work focused on long spans with relatively large displacements. Psimoulis and Stiros (2013) used RTS and TPS to estimate low amplitude displacements (between 2.5 and 6 mm, 0.1 and 0.24 inches respectively) for short-span bridges under trains. Watson et al. (2007) and Nickitopoulou (2006) proposed that the use of GPS for displacement estimations is a promising approach. Nassif et al. (2005) proposed using remote monitoring with a laser Doppler vibrometer. Other studies identified simplified methods relating dynamic measurements of inclinometers with deflections (Hou et al. 2005). Each of these approaches has its limitations, including cost and complexity, which has prevented widespread use.

Accelerations have also been explored in the past as a convenient means to estimate displacements. Accelerations do not require a fixed reference point from which to measure, and accelerometers can be easily installed and removed from bridges in the field. The most common approach for estimating displacements from acceleration is to use double integration, with an adjustment to eliminate the drift caused by integration constants (Iwan, 1985; Boore, 2003; Yang et al. 2005; Gindy et al. 2008). In general, such methods require initial condition information and are not necessarily suitable for structures such as railroad bridges, which are dominated by low frequency response components.

2.13 Fragility assessment

The fragility of a structural system is defined as the conditional frequency of failure if a given input parameter of value is applied to this system (Mosleb and Apostolakis, 1986; Shinozuka et al. 2000). Fragility curves can relate the variability of bridge serviceability conditions associated to a given level of displacement (Figure 2.20).

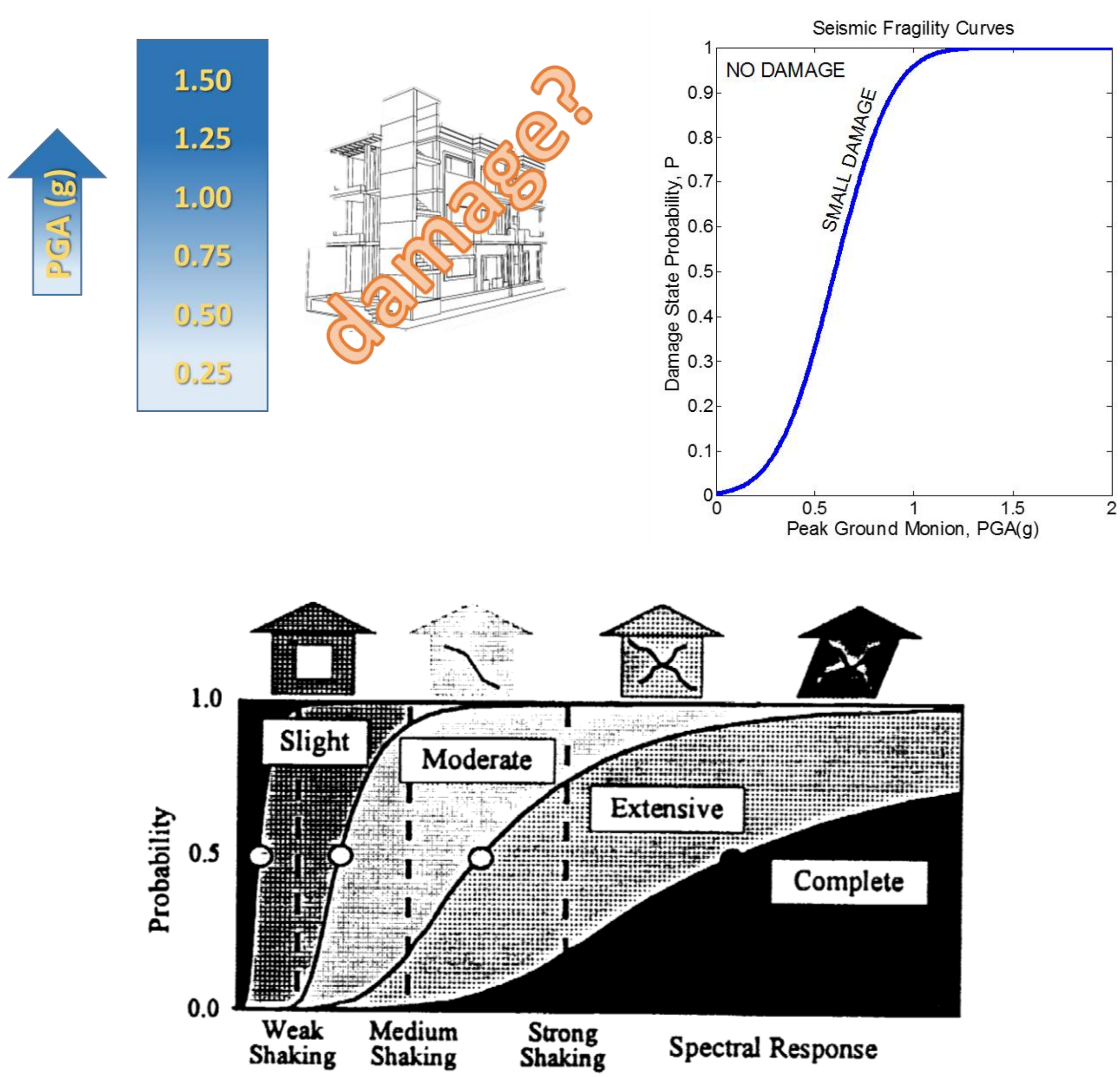


Figure 2.20. Fragility curve example for seismic engineering.

Originally developed in the context for nuclear engineering industry (Siu and Kelly, 1998), seismic engineering fragility curves relate ground motion intensity levels to the probability of experiencing damage levels/states (Singhal & Kiremidjian, 1996):

$$P\left[X \le x\right] = \int_{z=-\infty}^{x} f_x(z)dz \tag{2.5}$$

If the variables are taken as discrete values, then

$$P\left[X = x\right] = p_x\left(x\right) \tag{2.6}$$

, and the cumulative distribution function is calculated as:

$$P\left[X \le x\right] = \int_{z=-\infty}^{x} p_x(z)dz = \sum_{z=-\infty}^{x} p_x(z) \tag{2.7}$$

For the case of estimating the fragility of one building under earthquake (Figure 2.21), we can obtain the cumulative probabilities of exceeding a given service limit as shown in Table 2.3. For computational purposes, parameters for lognormal distributions can be calculated for each fragility curve as independently estimated under the maximum likelihood.

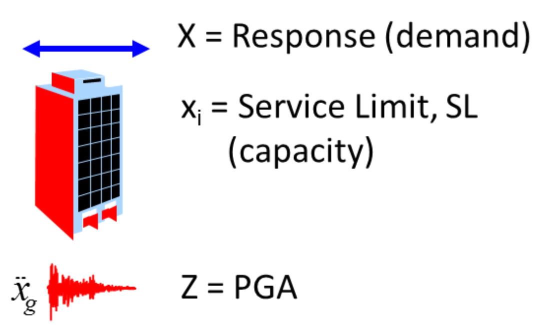


Figure 2.21. Fragility framework for seismic demand of buildings.

Table 2.4. Fragility example for a given seismic demand.

PGA	0.25g	0.5g	0.75g	1.0g
SL 1 (X<x1)	20/100	60/200	20/50	1/10
Probability	0.2	0.3	0.4	0.1
Cumulative	0.2	0.5	0.9	1.0

Once multiple fragilities are obtained, the probabilities of service limits using fragility curves are the different probabilities of service limits given hazard levels. The service limits are mutually exclusive (Figure 2.22). For a given set of four fragility curves, this can be written in the form:

$$P_{i1} = P\left(a_i, SL_1\right) = 1 - F_1\left(a_i\right) \tag{2.8}$$

$$P_{i2} = P\left(a_i, SL_2\right) = F_1\left(a_i\right) - F_2\left(a_i\right) \tag{2.9}$$

$$P_{i3} = P\left(a_i, SL_3\right) = F_2\left(a_i\right) - F_3\left(a_i\right) \tag{2.10}$$

$$P_{i4} = P\left(a_i, SL_4\right) = F_3\left(a_i\right) \tag{2.11}$$

, where

$$F_j\left(a_i\right) = P\left(\delta_{Di} > \delta_{Cj} \middle| a_i, \delta_{Cj}\right) \tag{2.12}$$

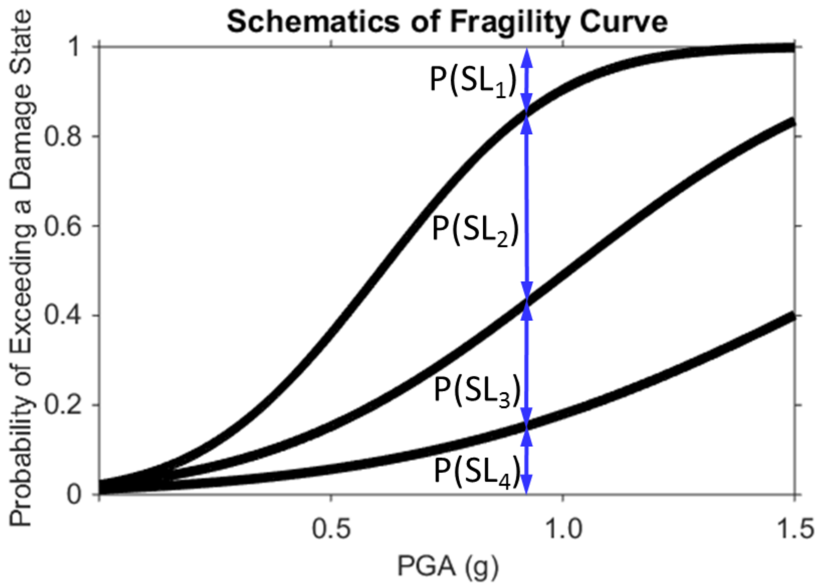


Figure 2.22. Probability of service using fragility curves.

Fragility curves can be formulated using experts opinions, empirical data, analytical data, or hybrid data, which combines the data from multiple sources (Li et al. 2012) (Figure 2.23). Benefits of building and using fragility curves include its simple visualization which assists in quickly comparing and observing differences between fragilities of different structures, which makes fragility curves a very practical tool for assessing damage-motion regional loss (Anagnos et al. 1995).

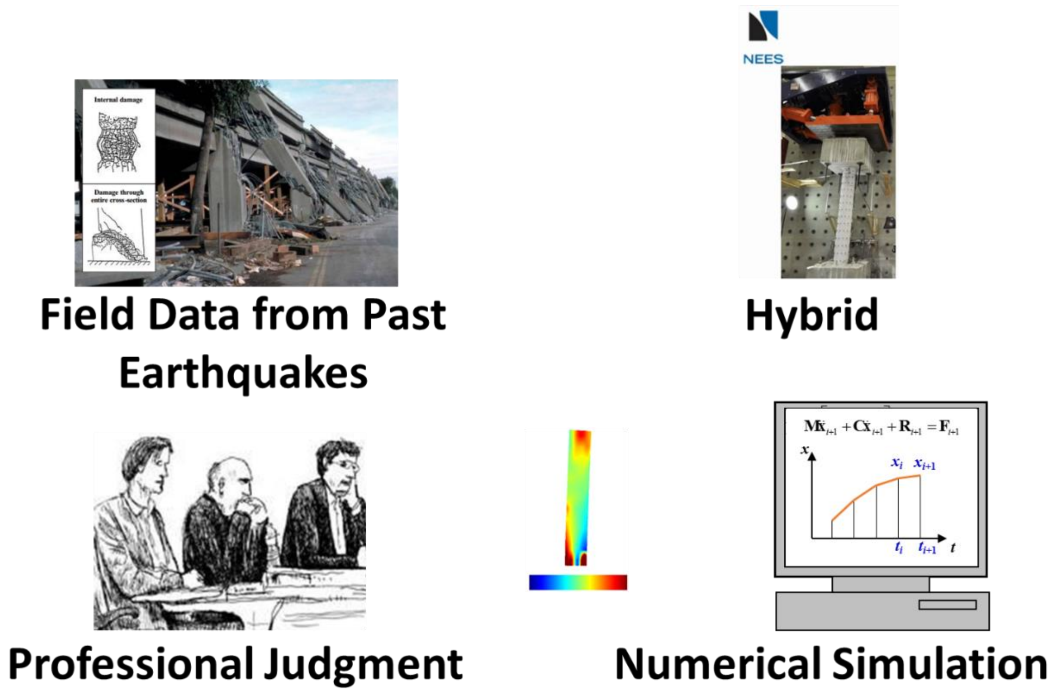


Figure 2.23. Types of sources for fragility curves.

In the seismic case, the Federal Emergency Management Agency (FEMA) funded a seismic study by the Applied Technology Council (ATC) that resulted in the ATC-13 report, entitled Earthquake Damage Evaluation Data for California (Rojahn and Sharpe, 1985). This report presents expert-opinion earthquake damage and loss estimates for California infrastructure in the form of matrices. The report also describes methods to estimate losses on a regional basis. Anagnos et al. (1995) described how to convert the results obtained in this report in statistical means to provide fragility assessments using experts' opinions. This approach related the Modified Mercalli Intensity (MMI) as the input (specified level of ground motion) and the levels of damage where defined with simple definitions, which were selected by the experts for different types of building structures. The probable distribution of their answers provided the relationship between the input (ground motion) and the consequence. Today, even when the displacement is accepted to be a clear indication of the railroad bridge condition, there is not a document that relates the levels of displacements to railroad bridge safe and cost effective operations. Past studies of railroad bridges fragilities from analyzed deck plate girders structural capacity (Park and Choi, 2011). However, their emphasis is not the serviceability using measured displacements.

Railroads want to estimate expenses to prioritize their consequence-based decisions. Railroads can improve MRR decisions at the network level using objective data collected from the field and comparing their relative impact at the network level. Currently, bridges make MRR decisions relaying on bridge inspection reports generated annually (AREMA, 2008, 2014). In fact, researchers have identified that in addition to bridge reliability quantification, BMS need to integrate minimum expected maintenance costs over a given period of time, assisting owners to optimize their future use of their budgets (Frangopol et al. 1997; Frangopol et al. 1999). Additionally, Padgett et al. (2010) conducted regional seismic assessment of a highway bridge network in Charleston, South Carolina and their associated costs for an entire network ratios. Furthermore, Li et al. (2012) used fragility curves of Imperial County, California, to estimate network repair costs associated to an Earthquake scenario. The different probabilities associated to the different levels of damaged were quantified at the network level. However, railroads are interested in measuring bridge performance to inform their decisions, a technical tool combining bridge measured performance, service levels and operational expenses is still lacking.

CHAPTER 3 SURVEY-BASED STUDY: CURRENT RESEARCH NEEDS IN RAILROAD BRIDGES AND STRUCTURAL ENGINEERING

Railroad bridges are different from highway bridges and need to be maintained following different considerations. However, there is little work developed in current literature that address railroad bridges challenges individually or specifically. In order to develop a new study that address specific needs of railroad bridges, a survey-based study was conducted to identify which are the specific current research needs in railroad bridges and structural engineering in US. This chapter describes the details of this survey-based study as well as the main results.

3.1 Description of needs

On October 28 and 29, 1987 the University of Illinois hosted a workshop entitled “The National Workshop on Railway Bridge Research Needs.” The objective of that workshop was to identify the most important research topics regarding railroad bridges and structural engineering. The following literature review outlines research projects conducted as a consequence of the workshop (Groskopf, 1990; Anonymous, 1994). Details about the organization of the workshop, the list of topics selected and discussed, and a summary of the findings can be found in a report published by the AAR (Foutch, 1989).

Today, the need for a new “meeting” to identify current research needs in North America is overdue. While a North American Workshop on Railroad Bridge Research Needs should be planned and organized, a survey of national experts on railroad bridges and structural engineering has been conducted in the meantime to best identify current topics for railroad bridges and structural engineering research.

3.2 Survey results: NSEL comprehensive report

Details about the execution of the survey are available in the Newmark Structural Engineering Laboratory (NSEL) report entitled "Current Research Topics: Railroad Bridges and Structural Engineering" (Moreu and LaFave, 2012), including detailing about the interviewing chronology and related procedures. The results of the survey are listed in Table 3.1. Comparison of research topics on railroad bridges between 2010-11 and 1987.

Table 3.1. Top research needs of structural engineering of railroad bridges.

2010-11 TOPICS	2010-11 RANKING	1987 TOPICS	1987 RANKING
Deflection measurements	1	Determining loads in the field	1
High speed trains	2	Investigate impact factor and effects	2
Long-span bridges	3	Fatigue life	3
Approaches	4	Determine longitudinal forces	4
Longitudinal forces	5	Develop better analysis for design	5
New design loads	6	Timber non-destructive testing	6

This survey-based study has compared the results of this survey-based study to those of the 1987 NSF Workshop (based on the paper by Byers and Otter from 2006 collecting the results, and the priorities identified at that time). That comparison clearly illustrates the evolution of terminologies and topics between 1987 and 2010-11, further validating the need for this new survey-based study. This side-by-side comparison acknowledges that there are some similarities between research areas from over 20 years ago and today. For example, both studies identify successfully and economically making various types of railroad bridges field measurements as a high research priority. Table 3.1 presents the results of this comparison.

The current survey has identified the need to approach the bridge design, construction and management from a strictly economic point of view as the most important particularity governing structural engineering in railroad bridges. The chief governing need for engineers is to assure the structural integrity of their railroad bridges in use and to communicate their actual structural capacity within personnel and departments within the railroads. Decisions regarding railroad bridges and structural engineering in design, management, maintenance, and construction should always be made to reinforce the safety of railroad operations. Designing, building, and maintaining railroad bridges must be directed from an economic view, to ensure the safety of railroad operations.

3.3 Conclusions: current research topics in railroad bridges and structural engineering

A survey of sixteen structural engineers has been conducted. The combined experience of the sixteen interviewees in railroad bridges and structural engineering added up to more than 500 years. The goal of the survey was to identify the main structural engineering topics for railroad bridges today. Consultants, contractors, federal officers, and railroads were interviewed during the course the survey. Both experienced engineers and entry-level personnel were questioned about their opinions regarding several research topics involving railroad bridges and structural engineering.

According to this survey-based study, determining the capacity of bridges that are in service has been identified as the top responsibility and concern of the engineers in charge of railroad bridges – assessing the performance of railroad bridges in the field under real railroad traffic to allow more objective decision making. Investing in maintenance tools that can assist in improving bridge capacities once they have been assessed has also been identified. Quick replacements and member prioritization are of interest, too, so research of new materials and construction methods are valued. Finally, the design of alternatives for future demands like HSR (AREMA, 2010) and Heavy Axle Load (HAL) (Otter and Joy, 2010), and the need to replace bridges that are over 100 years old, are other priorities for railroad bridge structural engineers today.

The promising future of freight and passenger railroad traffic in the United States needs to be seconded by an upgrade in the railroad infrastructure supporting it. This improvement and development of the railroad infrastructure will need to address in particular the most complex elements of the railroad infrastructure, their bridges. A growing railroad industry needs to be supported by healthy and robust bridges. Economic and safety considerations are both concerns of the railroad industry in the United States. A robust, reliable network is safer, more efficient, and therefore more productive.

An overall thrust of general interest was to approach bridge design, construction, maintenance, and management from an economic point of view. According to this study, and in light of new federal regulations for railroad bridge management published by the FRA (2010b), future research could be directed toward better enabling the assessment of bridge capacity. A major responsibility and concern of bridge engineers in charge of railroad bridges in North America today is assessing the structural performance, response, and/or decay of those bridges under both: (a) regular loading conditions (long term assessment), and (b) unusual and/or unexpected events (collision, severe scouring, etc.).

This survey-based study ranked measuring deflections under live loads as the current top research interest. According to the majority of the engineers in the survey, measuring real-time deflections under live loading can be beneficial both in terms of railroad bridge management and railroad bridge replacement prioritization, especially for timber bridges. With measurements of accurate bridge performance-related parameters, such as displacement, railroads could direct their annual budgets to only replacing those bridges most in need.

This survey-based study found the potential impact of high-speed trains on current and future railroad bridges to also be of high priority. Interviewees identified this topic as one of growing interest due to the foreseeable need for this research in order to properly accommodate high-speed traffic in North America. In their opinion, certain existing bridges would have to be upgraded or completely replaced in order to accommodate passenger trains with higher speeds. This study gave some priority to advancing the knowledge about long-span railroad bridge design, based primarily on the forecasted need of replacing existing longer-span bridges at major elevated crossings that were designed and constructed more than 100 years ago (which sometimes now have significant maintenance costs). Bridge engineers further expressed interest toward research about the maintenance of existing deteriorating bridge approaches, as well as techniques and methods to design more durable railroad bridge approaches in the future. And finally, this survey-based study of engineers placed the examination of longitudinal loads in railroad bridges (their magnitude and distribution, including design implications), as well as the general need for research that develops better design loads and methods for new railroad bridge design, as two other quite important research needs.

Other research topics identified during the survey and parallel literature review were suggested by the railroad bridge structural engineers as areas of some interest for further consideration. Railroad bridge structural engineers generally prioritized investing in whatever maintenance tools could assist them in measuring and/or improving bridge capacities. This group of experts identified the need for developing new methods of measuring bridge foundation capacity during and after scouring events. Additionally, quick bridge replacements and member replacement prioritization are of emerging interest, as are research on new materials and construction methods.

3.4 Future research steps and recommendations: Structural Health Monitoring and railroad bridges in the US

Efforts towards railroad bridge structural engineering field assessment have been recognized as a main interest of structural engineers to improve inspection and maintenance operations. Bridge assessment and monitoring will also benefit and improve new bridge construction control, as well as bridge replacement prioritization. Research must be directed to areas that can assist toward prioritizing railroad bridge replacements, and to implement intelligent and efficient decision-making tools in the railroad bridge industry. Objective data collection in the field can help quantify bridge structural capacity and provide a structural engineer with ways of more efficiently determining which bridges and/or bridge elements to replace under a limited budget. Determining the capacity of existing timber trestles (still in significant use in the United States today) can benefit from this. Construction activities can also be improved from data collection, in order to protect existing structures from adjacent construction operations. New means and methods, technology, and materials will assist in quick bridge replacements. The railroad engineering community should promote interdisciplinary collaborations between different engineering areas to incorporate new technologies that can assist and develop inexpensive tools that are easy to install and read, such as wireless sensors. The recent proliferation and development of these new data collection sensors (wireless sensors for SHM) in the last 10 years, along with pilot experiences presented in this study, identify this as an area that should be researched in the near future by railroad bridge structural engineering institutions and affiliated laboratories. This is, incidentally, quite similar to something recently called for as a particular area for research and development in the AREMA President's column (Unsworth, 2011).

CHAPTER 4 FRAMEWORK TO INFORM DECISIONS FOR THE MANAGEMENT OF RAILROAD BRIDGE NETWORKS

This chapter develops an approach for consequence-based management of bridge networks for making MRR decisions on a network-wide basis. Current MRR decisions of railroad bridges are informed by bridge inspections and ratings. Inspection and rating practices recommend observing the response of bridges under revenue traffic. However, an objective relationship between bridge responses, bridge service state condition, and the associated impact to railroad operations has yet to be established. As a result, current MRR decisions are in general conservative, prioritizing decisions to overcome the uncertainty of consequences of inaction. If the consequences of MRR decisions could be better determined, then the railroads could more effectively allocate their limited resources. This framework provides a consistent approach for intelligent management of railroad bridges, and more specifically, for the prioritization of railroad bridge MRR decisions. Using this framework the rail owner can identify the most efficient use of a limited budget while maintaining safe railroad operations. The following subsections describe the overall layout of the framework followed by a detailed description of each of the six components.

4.1 Introduction

This proposed consequence-based approach for the management of railroad bridge networks is comprised of six components, as shown in Figure 4.1. The first component is the hazard, assumed as the maximum transverse displacement of bridges in the work plan, measured under a loaded train running at the maximum allowable speed for their track class. The second component of the framework is the inventory. The bridges in the inventory belong to the work program, and have already been identified by the railroad to need MRR decisions. The framework will inform how to prioritize MRR decisions of bridges within this inventory. The third component are fragility curves elaborated assuming that bridges within the inventory (component two) have similar structural properties and that the serviceability of each bridge is independent from the bridge location within the network. In this first layout of the framework, the service levels of two contiguous bridges are independent of each other. The fourth component uses the maximum measurement of displacement for a given bridge under trains, assuming this maximum displacement represents the bridge condition, following indications from the railroad. The fifth element calculates the operational costs per year assuming operational expenses of unplanned engineering work provided by the railroad as well as lost revenue related to delay or interruptions to traffic. The sixth component assumes that the operational costs related to the conditions of the bridge are the only variables in the decision making, neglecting other factors including, but not limited to: access to the bridge, financial decisions related to strategy planning of operations, proximity of related railroad operations to the bridge, etc. The following subsections describe each of the six components in detail.

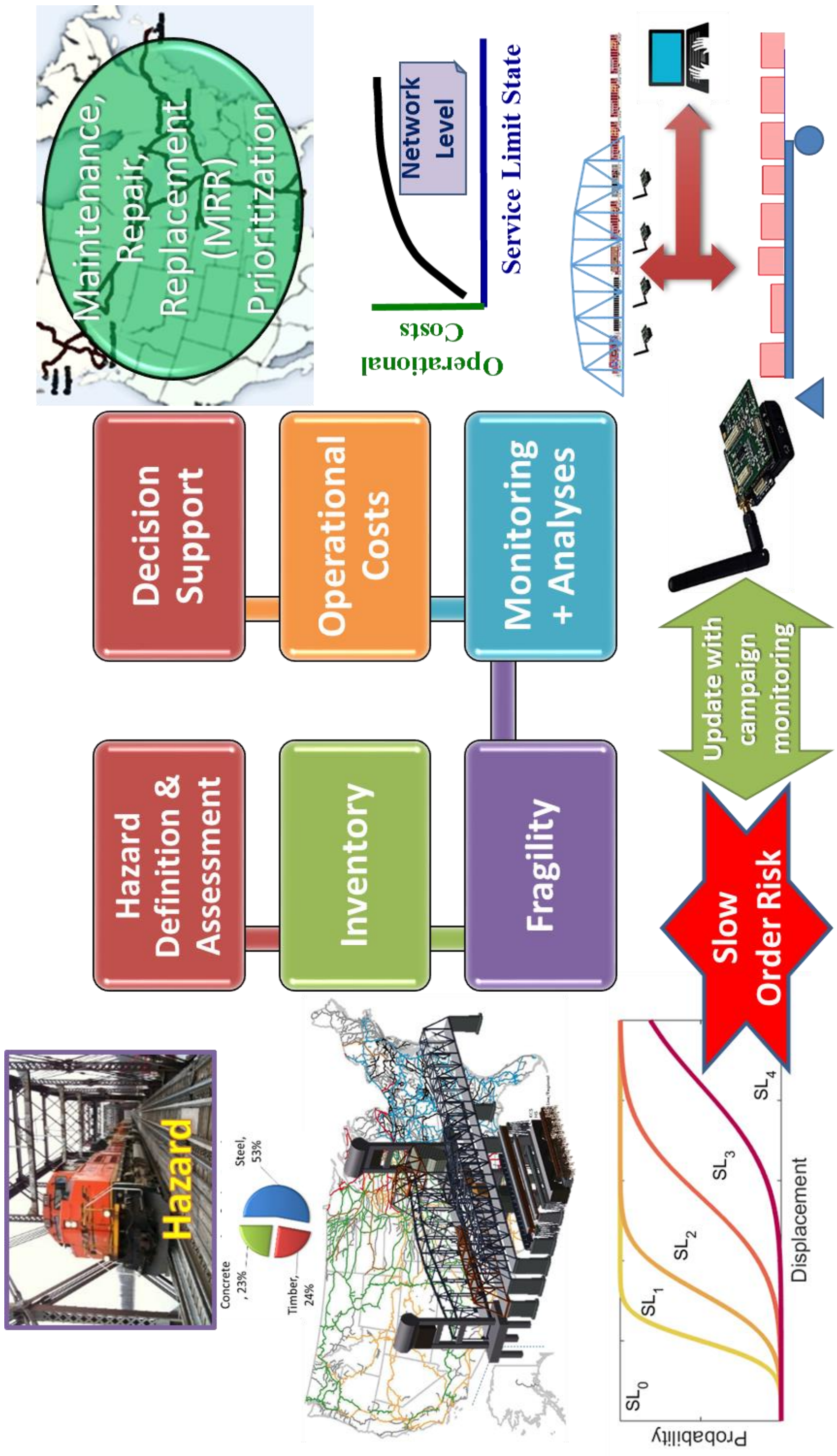


Figure 4.1 Framework for risk-based management of railroad bridge infrastructure.

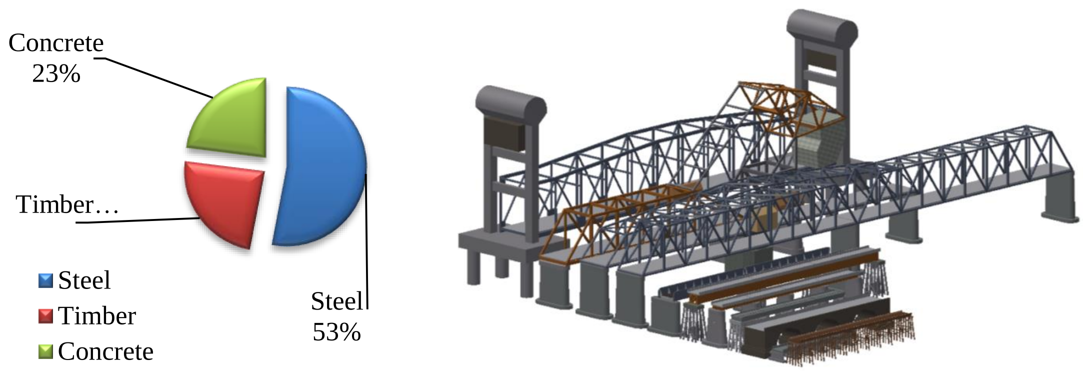
4.2 Hazard

The first component is the framework is the hazard. In the context of earthquake engineering, the hazard is characterized by some measure of the magnitude of an earthquake (i.e., peak ground acceleration or PGA). Fragility relations are then used to relate the PGA to the likelihood of a structure being in a certain damage state after the event. In the case of the railroad bridges, the train is the primary “hazard” or loading to the bridge. Moreu et al. (2014) found that bridge displacements can provide an important indication of the service level (or state) of railroad bridges. Therefore, for this framework, the train crossing event will be considered the hazard, and the bridge displacement under revenue service traffic will be the metric measuring the hazard.

4.3 Inventory

The second component is the inventory, which corresponds here to the population of bridges owned by the specific railroad for which the MRR policies are being developed, and their current structural condition. The bridges in the inventory belong to the work program, and have already been identified by the railroad to need MRR decisions. This framework assumes that the bridges being monitored share similar structural properties and operational concerns, so that the measurement of the hazard of different components of the inventory can be used for relative comparisons within the inventory. Figure 4.2(a) shows a classification of North American railroad bridges showing percentages by length and material type (FRA, 2008). Figure 4.2 (b) shows a more detailed classification of the eleven railroad bridges types based on superstructure materials and structural type (Moreu et al. 2012). This classification of railroad bridges in the US was developed in accordance with past railroad bridge classification efforts (AREMA, 2008; Sorgenfrei and Marianos, 2000; International Heavy Haul Association (IHHA), 2009; Parsons

Brinckerhoff Quade & Douglas, Inc., 1980; ENSCO, 1994). A list of railroad bridge structural engineering concerns can be assigned for each specific railroad bridge type. The current information of each bridge is provided by the railroad company owning the bridges, based on the most recent bridge annual inspection (required by bridge safety standards and the FRA (2010). In this initial effort for establishing this framework, the relative importance of the bridges is assumed equal, in order to prioritize the differences in serviceability to inform MRR decisions.



*Steel includes iron, concrete includes masonry.

Figure 4.2 Railroad bridge population (a) from the most recent FRA survey (FRA, 2008), (b) by superstructure type.

4.4 Railroad bridge fragility curves

This research employs fragility curves to correlate bridge service condition to bridge displacements under revenue service traffic. Fragility curves are a statistical tool representing the probability of exceeding a given performance (or damage) state as a function of an engineering demand parameter. In this paper, service limit-states (SL) represent the consequences to rail operations associated with bridge displacement. This framework proposes five different SL of railroad bridge serviceability using bridge performance under trains. Freight trains can be conservatively assumed to have the same weight, whereas their interaction during train crossing is different depending on multiple factors, such as the train speed and geometry and condition of both track and bridge (Hussain et al. 1980; FRA 2005; FRA 2012; Wolf, 2005; Watco, 2012). The

SL of this framework are described by railroads experts based on standard railroad bridge management decisions and are listed below, followed by their effect to railroad operations:

- SL_0 – No Action: this is the preferred state. If displacements are low, rail operations are safe and there is not a menace to serviceability-related problems. This limit state is required for completeness in the analysis.
- SL_1 – Inspection: the first decision when a bridge moves excessively under regular traffic and before traffic interruptions. Inspections typically include some minor maintenance work associated.
- SL_2 – Temporary Slow Order (TSO): if the movements are excessive, then the speed of trains is reduced with a TSO, associated to some small (local) maintenance/repair work.
- SL_3 – Permanent Slow Order (PSO): if the TSO does not address the serviceability of the bridge, a PSO is ordered to secure safe railroad operations until the bridge receives significant repairs, permanently slowing traffic over the bridge until it is upgraded (with MRR decisions).
- SL_4 – Track Outage (TO): when the bridge condition is not safe for train crossing, the bridge is put out of service (until the bridge condition is upgraded with MRR decisions and the bridge is ready to carry trains again).

Figure 4.3 provides a conceptual representation of the inherent variability between bridge displacements and the five SLs. This variability is due to several issues, including: imprecision in the bridge service limit-states, differences in train weights, changes in foundation stiffness due to weather and/or seasonal changes, track and vehicle non-linear performance under different revenue service traffic, etc. Nevertheless, measuring displacements under traffic provide objective information about the service limit-state of the bridge.

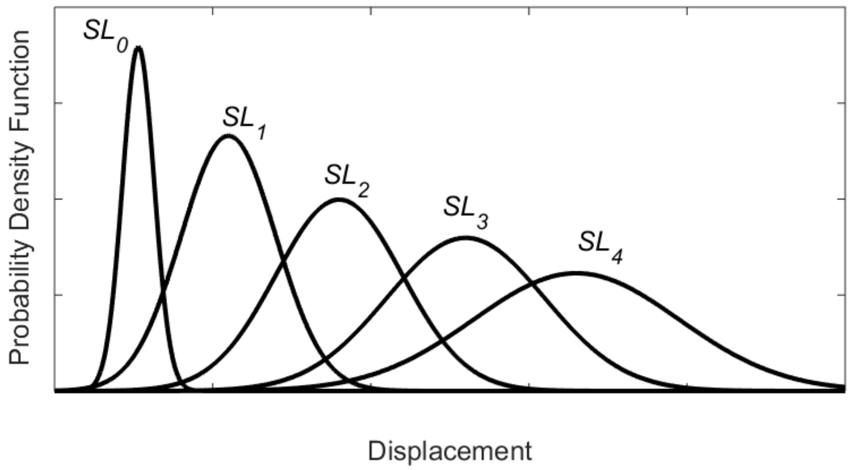


Figure 4.3. Probability density function of displacements for each *SL*.

North American railroads have different track classes corresponding to different traffic speeds (FRA, 2015), and studies by Moreu et al. (2014) show that displacements depend on train speed. Therefore different fragility curves are developed for each track class, assuming that trains are running at their maximum allowable speed. In the context of this research, the fragility function $F_{k,j}(d)$ is defined as the probability of being in service limit-state $SL \leq k$, given that the maximum displacement of the bridge is *d* for track class *Z* = *j*, i.e.,

$$F_{k,j}(d) = P\left(SL_k \middle| D = d, Z_j\right) \tag{4.1}$$

, where

D = random variable representing the maximum measured displacement of the bridge under revenue traffic,

d = realization of the random variable *D*,

SL_k = service limit-state, with *k* = 1, 2, 3, 4,

Z_j = track class, with j = 1, 2, 3, 4, 5.

Fragility curves are often fit by a two-parameter lognormal distribution (Nuclear Regulatory Comission, 1983; Shinozuka et al. 2000; Wen et al. 2003; Nielson et al. 2005) or:

$$F_{k,j}(d) = \Phi\left[\frac{\ln\left(d/c_{k,j}\right)}{\zeta_{k,j}}\right] \tag{4.2}$$

, where

Φ = cumulative distribution function of the standard normal distribution,

$c_{k,j}, \zeta_{k,j}$ = parameters of the lognormal distribution representing the fragility curve.

In this paper a fragility curve for a particular railroad bridge SL_k is obtained by computing the conditional probabilities of a given SL_k being exceeded. For example, fragility curves for SL_2 describe the probability of requiring a TSO given a measured displacement. Figure 4.4 shows one example of fragility curves of all SL_k for one specific track class Z_j.

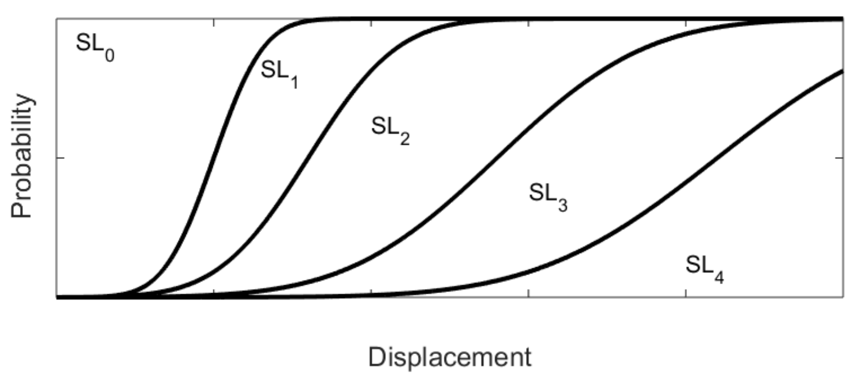


Figure 4.4. Conceptual depiction of railroad bridges fragility curves $F_k(d)$ for different SL_k for one specific track class Z_j.

To date, the data necessary to build probability distributions for displacements of a bridge in a given SL_k is unavailable. To this end, experts are surveyed, giving them a specific maximum bridge displacement and asking them, based on their experience, to predict the most likely SL_k associated with this displacement. The probability associated to SL_k is proposed following the total probability rule by Ang and Tang (2007), see Figure 4.5. The probability for each SL_k region $P\left(SL_{k,j}\right)$ then can be approximated using the distribution of the expert's answers:

$$P\left(SL_{k,j}\right) = F_{k,j}\left(d\right) - F_{k+1,j}\left(d\right) \tag{4.3}$$

where

$SL_{k,j}$ = service limit-state and j track class, with k =0, 1, 2, and 3,

$F_{k,j}$ = fragility curve for the k^{th} SL and j track class,

$F_{0,j}$ = 1, for all j track class,

and

$$P\left(SL_{4,j}\right) = F_{4,j}\left(d\right) \tag{4.4}$$

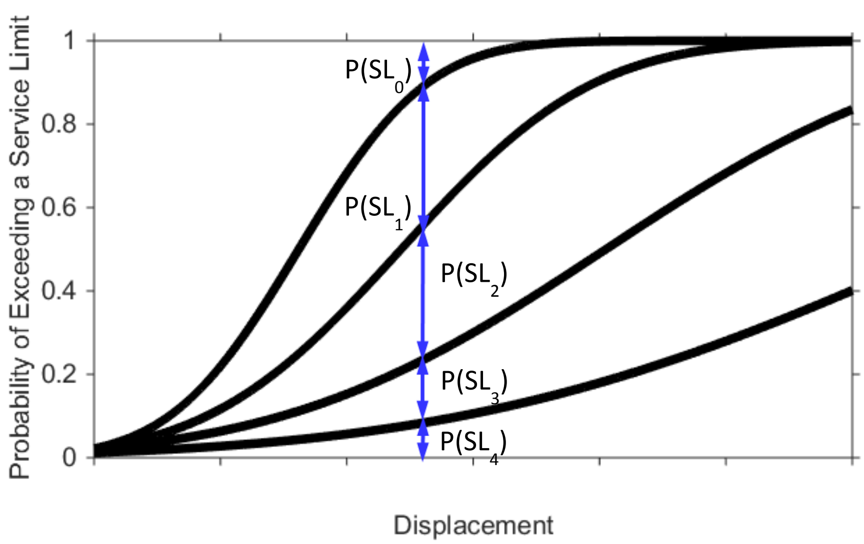


Figure 4.5. Probability of Service Limit States Using Fragility Curves.

4.4.1 Slow order fragility curve definition

Fragility curves for a slow order limit state describe the probability of requiring a slow order given a measured displacement for a bridge of certain properties predefined and for trains of the same weight and speed (track level). The distribution of this service limit state has a normal distribution with mean equal to 1 inch, and a variance of ½ inch. The consequences of displacement on rail operations can be identified by different methods: analytically, using field data, or using expert opinions. Analytical fragility curves for timber railroad bridges are computationally challenging and expensive. Moreu et al. (2014) identified that for a timber railroad bridge of 20 ft, a displacement of 1 inch indicated the need for possible bridge management action. This value can be upgraded with more field measurements of similar bridges of different conditions. While collecting data in the field when the bridge conditions are known is the best resource, experts can provide a preliminary relation between bridge displacements and bridge condition. Figure 4.6 shows the result of plotting the conditional cumulative probability of a class II track being changed to class I track. Similarly, the same could be done with the different track class levels, and these curves can be updated using Bayesian theory (Jian et al. 2013).

Figure 4.6. Fragility curve of bridge condition based on measured displacement under trains.

4.4.2 Seismic fragilities vs. railroad fragilities

As opposed to seismic fragilities (Figure 4.7), where the estimated demand is used to calculate the probability of a service limit state, the railroad fragilities estimate the capacity using the measured response (displacement under train) (Figure 4.8). The comparison between the two different fragilities is provided in Table 4.1. Railroad performance fragilities use measured displacements to estimate unknown capacity of the bridge.

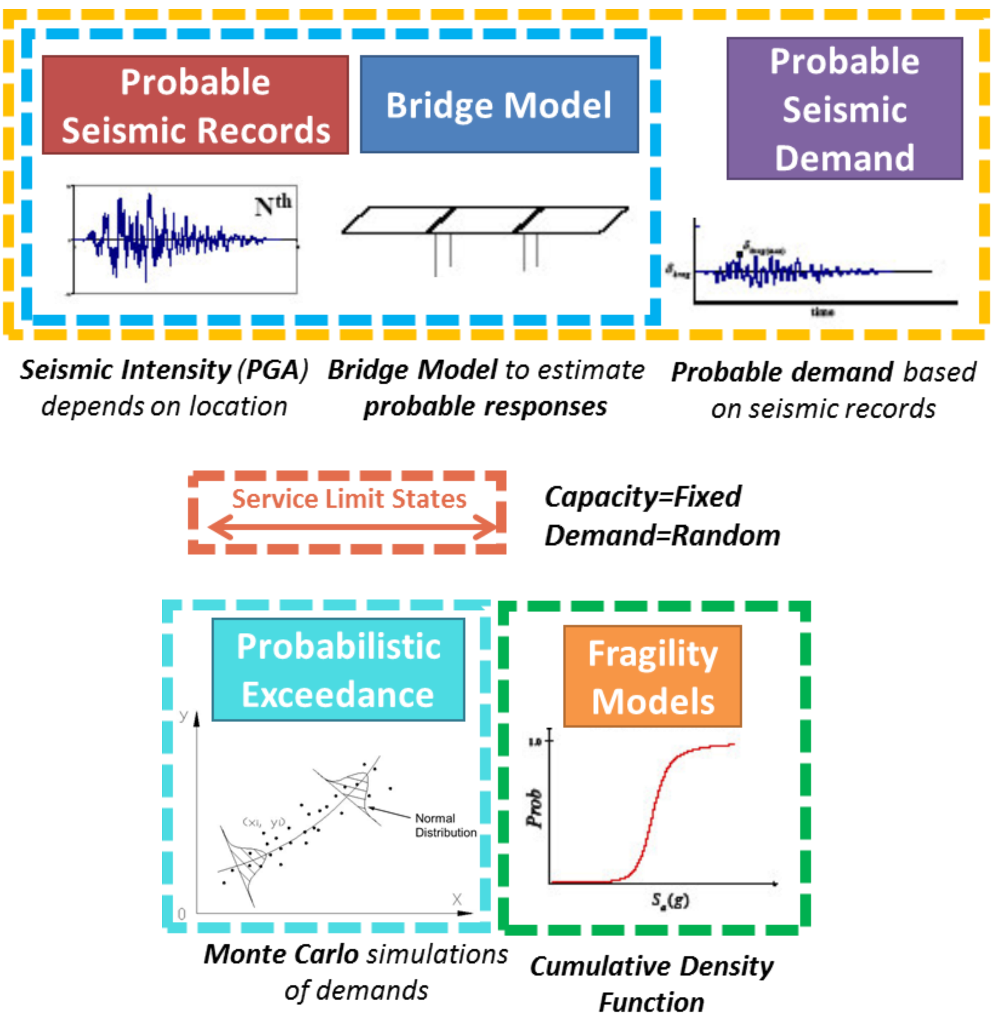


Figure 4.7. Seismic fragility.

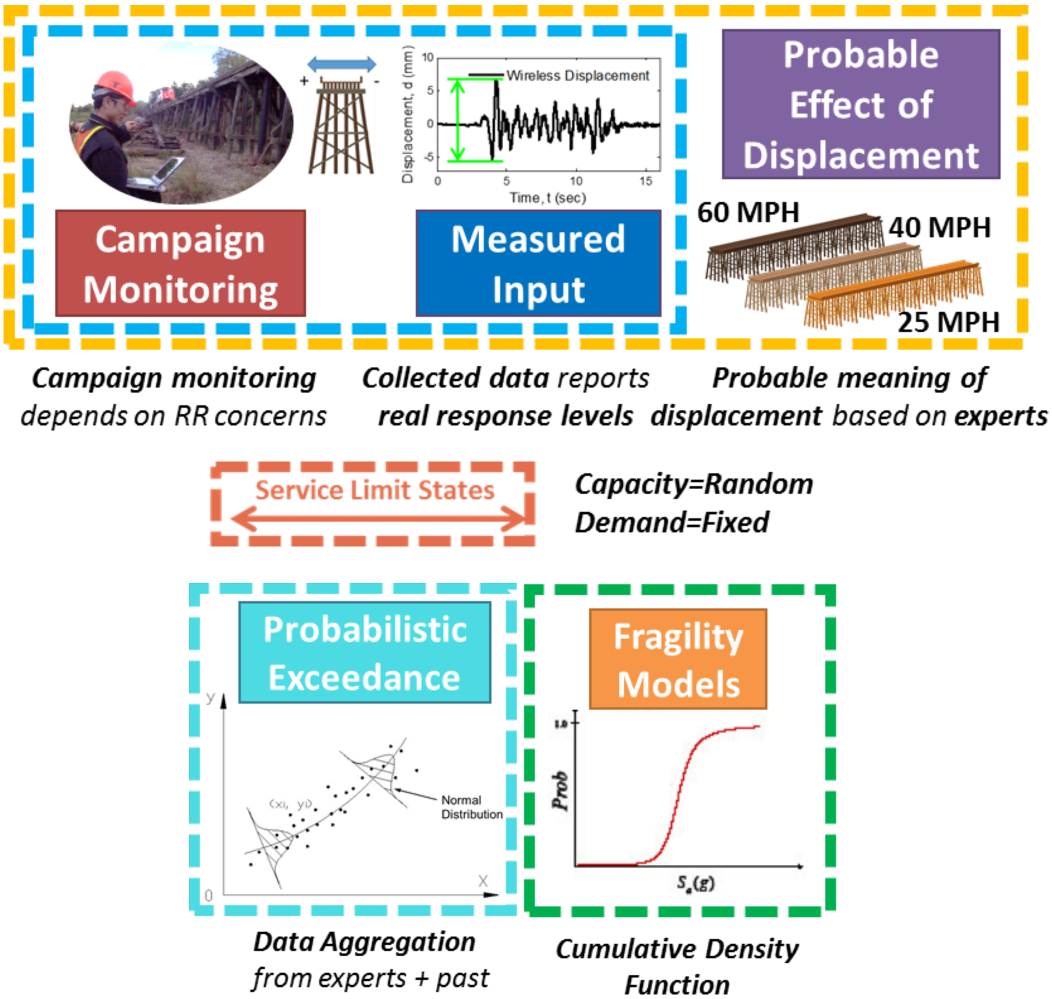


Figure 4.8. Railroad fragility.

Table 4.1. Comparison between seismic and railroad fragilities.

Comparison	Probable Seismic Damage	Probable Menace to Rail Operations
Application	Estimate probable future earthquake responses	Use today's real responses and probable meaning
Assumption	Dynamic model of structure	Train load is constant
Limitation	One model represents thousands of bridges	Measurements captures all the problems of one bridge
Uncertainty	Effect of PGA to seismic demands	Effect of bridge displacements to rail operations
Variability	Different seismic records cause different demands	Different bridges have different capacities
Input	PGA	Displacement
Output	Probable displacement estimated with simulations	Probable effect on railroad operations
Capacity	Determined by code/design	Determined by experts' opinion
Fragilities	Seismic structural damage	Menace to rail operations

4.5 Updating with campaign monitoring data

The forth component of this framework updates the relationships between bridge response and limit states by collecting data in the field of bridges of known condition. Current advances in sensing technology now permit railroad managers to collect bridge displacements under trains using wireless smart sensors in almost real-time, inexpensively and effectively. Wireless Smart Sensors (WSS) can provide reference-free displacements of multiple bridges with moderate effort, and these measurements can inform of bridge condition and provide evidence to inform prioritizing or delaying MRR decisions (Moreu et al. 2015). The new information provided by collecting data from bridges under known conditions can also be used to update the conditional probability of reaching a certain SL_k given the measured performance parameter (Li et al. 2012). This framework uses the maximum displacement of one bridge under revenue traffic collected in the field, d_{measured}.

Using d_{measured} the probability for each SL_k region $P\left(SL_{k,j}\right)$ is determined for each bridge individually. This framework assumes that the maximum displacement measured under a loaded freight train is independent of the train crossing event, based on the fact that locomotive engines have similar weight range and past field monitoring live load tests by Moreu et al. (2014 and 2015). This framework proposes using WSS to collect the maximum measured displacement under revenue service traffic annually, or as often as the railroad wants to update their MRR policies. For example, bridges within the network of higher concern would need to be monitored more frequently. Based on current railroad management practices, railroad bridges with performance concern should be tested every three months. Using the most current d_{measured} railroads can inform their MRR decisions based on objective information of each bridge.

Using Bayesian updating, displacements collected from bridges of believed service state can inform pre-stablished fragility curves, and update probabilities specific to each bridge. The Bayesian approach provides the updated probability of a random variable using data collected annually. The parameters describing the probability of bridge condition provided by experts' opinion is the starting state of knowledge. The new information is provided by using data collected from bridges during the annual inspection. This data provides a distribution of displacement based on the believed current state of the bridge. Following Ang and Tang (2007) formulation for the posterior distribution, the updated probability can be written as a function of the prior distribution, P^{prior} and the posterior realization, P based on the measured displacement $d_{\text{measurement}}$ (see Figure 4.9):

$$P^{\text{updated}}\left(SL=k\middle|D=d_{\text{measured}},Z=j\right)=\alpha\cdot P^{\text{prior}}\left(SL=k\middle|Z=j\right)\cdot P\left(D=d_{\text{measured}}\middle|SL=k,Z=j\right) \quad (4.5)$$

, where

d_{measured} = maximum measured displacement of the bridge under revenue traffic,

$P^{\text{updated}}\left(SL=k\middle|D=d_{\text{measured}},Z=j\right)$ = updated distribution function of SL given the measured displacement and the track class,

$P^{\text{prior}}\left(SL=k\middle|Z=j\right)$ = prior distribution of *SL*,

$$=\int_0^{\infty} P^{\text{prior}}\left(SL=k\middle|D=d,Z=j\right)\cdot P^{\text{prior}}\left(D=d\middle|Z=j\right)\mathrm{d}d \,,$$

$P^{\text{prior}}\left(D=d\middle|Z=j\right)$ = prior distribution of displacement, *d*, based on the current state of the bridge,

$P^{\text{prior}}\left(D=d_{\text{measured}}\middle|SL=k,Z=j\right)$ = distribution of displacements, given the track class and service limit state (see Figure 4), evaluated at $D=d_{\text{measured}}$,

α = normalization parameter.

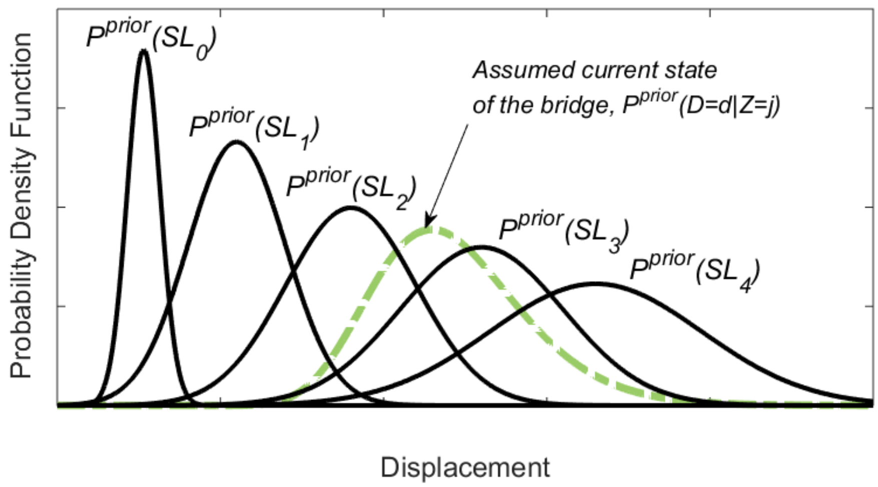


Figure 4.9. Probability distribution based on the believed current state of the bridge for updated bridge state assessment using measured data.

4.6 Annual operational costs

The fifth component of the framework is the relation between limit-states and operational costs to the owner for a given bridge. This framework relates service performance of bridges to operational costs that are not included in the annual work plan. Service limit-states are related to operational expenses depending on both the bridge type and the LOS. The associated operational costs for each SL_k are listed in Table 4.2. Expected expenses are calculated using general associated expenses to SL_k as provided for a Class I railroad, using confidential information from a representative territory within their network. For example, TSO and PSO limit-states have bridge engineering expenses related to estimates of maintenance and repair as provided by the railroad. Based on current experiences by the railroad, to correct a TO do not have significant expenses in bridge engineering or revenue, but the consequences to operations are large because they may not be identified ahead of time. To account for the safety concern of having TO at any given bridge, the bridge engineering expense and revenue lost are caused by the immediate repairs and the 180

PSO following the repairs. Using the fragility curves from the prior section, probable SL can be transformed in deterministic expenses for a given displacement and bridge.

Table 4.2. Operational costs of different *SL* based on unplanned bridge engineering expenses and lost revenues.

Service limit-state	**Decision**	**Operation Expenses, OE (%) (*)**	**Lost Revenue, LR**	
			days of slow order (d_{so})	**days of track outage (d_{to})**
SL_0	No Action	0	0	0
SL_1	Inspection	Fixed	0	0
SL_2	TSO	2	10	0
SL_3	PSO	5	365	0
SL_4	Track Outage (**)	10	180	2

(*) percentage of the total expense of replacement
(**) track outages expected expenses are augmented to include the negative consequences to operations of a non-detected track outage.

For each bridge, the annual operational costs OC for each limit state has two components: operational expenses, or the bridge engineering expense (i.e., the cost of MRR) OE and lost revenue expense LR (e.g., caused by slow orders or by track outages). The total operational cost OC for one bridge can be calculated as

$$OC = OE + LR \tag{4.6}$$

This paper estimates expenses assuming that the service limit states are mutually exclusive, as is done in seismic risk assessment (Shinozuka, 2000). Thus, the annual expected operational costs for one bridge can be calculated as:

$$< OC^n > = \sum_{k=1}^{K} P\left(SL = k \middle| D = d_{measured}, Z = j\right) \cdot OC_k^n \tag{4.7}$$

where

$< OC^n >$ = annual expected operational costs of each bridge *n*,

SL_k = service limit-state, with k = 1, 2, 3, 4,

OC_k^n = total expense for a bridge n in the k^{th} SL.

4.7 Consequence-based management to inform MRR decisions

The sixth component informs how to prioritize MRR decisions by minimizing total expenses to the network. The specific constrained minimization problem seeks to prioritize MRR decisions across time by minimizing the total cost of planned cost and unplanned operational costs overtime. Each year, both expected operational costs and MRR decisions must be kept under an annual maximum budget value. This constrained minimization problem can be written as

$$\min \sum_{y=1}^{Y} \sum_{n=1}^{N} \left(< OC^n > + MRR^n \right)^y \tag{4.8}$$

$$\text{subject to} \quad \sum_{n=1}^{N} MRR^n \leq MRR_{budget}$$

, where

Y = total number of years,

N = total number of bridges,

$< OC^n >$ = expected operational costs per bridge,

OC_{budget} = maximum operational costs bridge network budget/year,

MRR^n = MRR costs per bridge,

MRR_{budget} = maximum allowed MRR bridge network budget/year,.

This component of the framework permits railroads to minimize total network cost at the network level. Using the information from the prior components, operational costs are calculated from multiple MRR policies for a given population of bridges. The proposed framework can be used to minimize operational costs for a given MRR policy, improving MRR budget decisions within the network. Consequence-based management can provide savings by quantifying the costs associated to service levels based on performance measurements.

CHAPTER 5 ASSESSMENT OF TIMBER RAILROAD BRIDGES CONDITION UNDER DYNAMIC LOADS USING DISPLACEMENTS

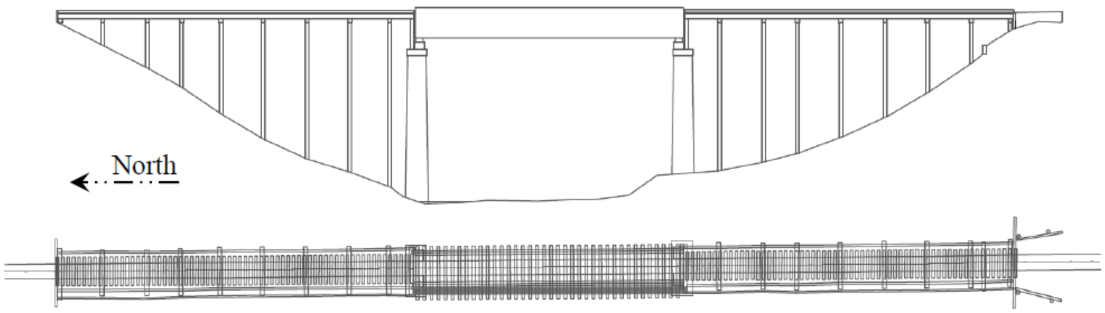
This chapter investigates how transverse displacements of timber bridges can be used to assess different bridge conditions for various traffic types. Vertical and transverse displacements of a timber bridge trestle pile bent have been collected and analyzed under different traffic conditions. Analysis of transverse bridge displacements in the time domain identifies the effects of train speed and direction on bridge performance. Analysis of transverse bridge displacements in the frequency domain shows evidence of harmonic roll. The research indicates that transverse displacements of timber trestles can provide a measure of bridge condition. For example, data from bridge monitoring campaigns which showed transverse displacements increased by up to three times during the construction process. Measuring and analyzing transverse displacements under revenue service traffic is shown to offer the potential for better condition assessment of timber railroad bridges.

5.1 Timber trestle monitoring experiment

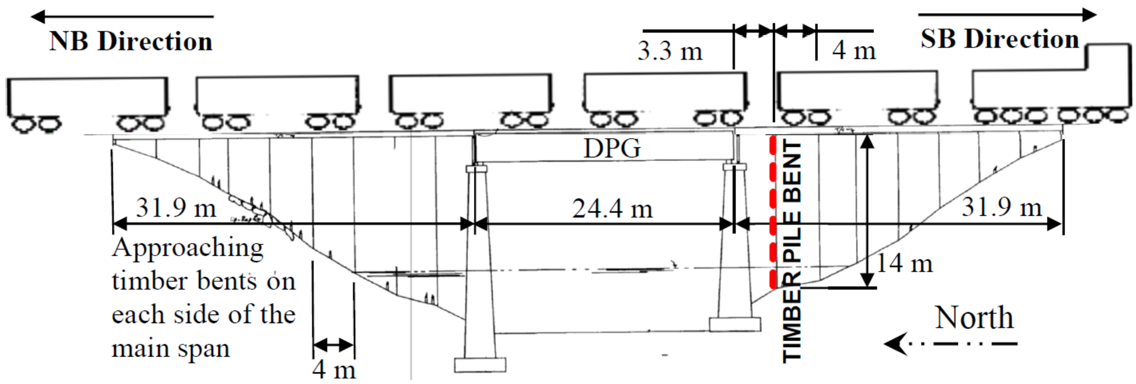
This section describes the bridge, the instrumentation, and the testing conducted as part of this research. Canadian National Railway (CN) scheduled a loading test on a timber trestle bridge approach using a Work Train (WT) with known geometries and loads. Vertical and transverse displacements and accelerations of the bridge were collected under the WT running at different speeds and directions, and also under revenue service traffic. By changing the loading conditions under otherwise known parameters, differences in transverse displacements could be associated with different bridge responses.

5.1.1 Bridge description

The Bluford bridge was along a side line of the CN railroad near Edgewood, IL, and consisted of one 24.4 m (80 ft) long deck-plate girder (DPG) supported by reinforced concrete piers, with eight and nine ballast timber deck panels on the South and North approaches, respectively. Figure 5.1(a) shows both the elevation and plan view of the overall bridge. The total length spans 88.2 m (289 ft) between the two abutments, with 0.16% track grade decreasing from North to South. Fourteen, 8 m (26 ft) long, stringers with their joints staggered on concrete caps, provided continuity over every bent, with a typical bent spacing of 4 m (13 ft). The foundations were timber piles driven below ground to an unknown length, probably to refusal based on railroad construction practices of the time. Soil borings available near the bridge show evidence of silty clay and silty sand at 9.1 m (30 ft) below ground, and poorly cemented sandstone as deep as 21.4 m (70 ft) below ground. Figure 5.1(b) is a general schematic elevation view of the bridge and its traffic directions. The maximum traffic speed allowed at the time of field testing was approximately 40 km/h (25 mph) due in part to on-going construction/maintenance work at the bridge. Figure 5.1(c) shows the South end of the bridge during instrumentation deployment.



(a)



(b)

(c)

Figure 5.1 Bridge views: (a) CAD elevation and plan view, (b) bridge dimensions and traffic, (c) timber trestle South approach: concrete pier, scaffold, and timber pile bent during sensor deployment.

5.1.2 Instrumentation

LVDTs (Linear variable differential transformers), as well as both wired and wireless accelerometers were installed on the 14 m (46 ft) tall pile bent located immediately to the South of the South concrete pier. A temporary scaffold (based on the ground, and braced to the adjacent concrete pier to increase its rigidity) provided a fixed reference point for making relative LVDT displacement measurements. An accelerometer was also installed on the scaffold, to indirectly infer its actual level of fixity.

Figure 5.2 shows the instrumentation of the pile cap, which included the following sensors:

- 2 wired uniaxial LVDTs (one vertical and one transverse, in the *X* and *Z* directions of Figure 5.2), for displacements;
- 1 wired bi-axial accelerometer atop the bent cap (in *X* and *Z* directions of Figure 5.2), for accelerations;
- 2 wireless tri-axial accelerometers attached to the bent cap (denoted as "1" and "2" in Figure 5.2); 1 wireless tri-axial accelerometer (denoted as "3" in Figure 5.2), attached to the scaffold to measure the relative "fixity" of the reference point under train-induced vibrations.

Figure 5.3 shows the relative location of the LVDTs with respect to the railroad timber trestle. This research uses displacement measurements of the timber piles under trains to assess the state the timber trestle bridge. The stringer's condition at the time of the experiment was not of concern, so they were not instrumented during the testing.

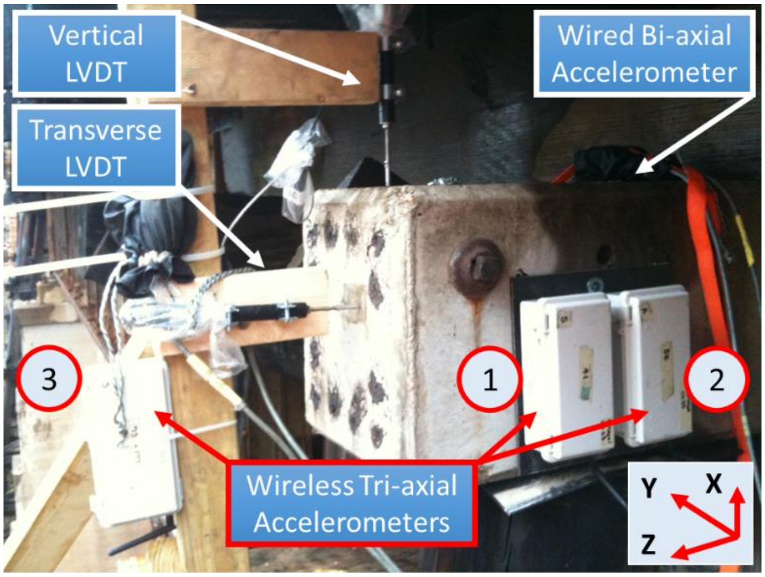


Figure 5.2. Pile cap instrumentation detail (showing LVDTs and accelerometers at pile cap).

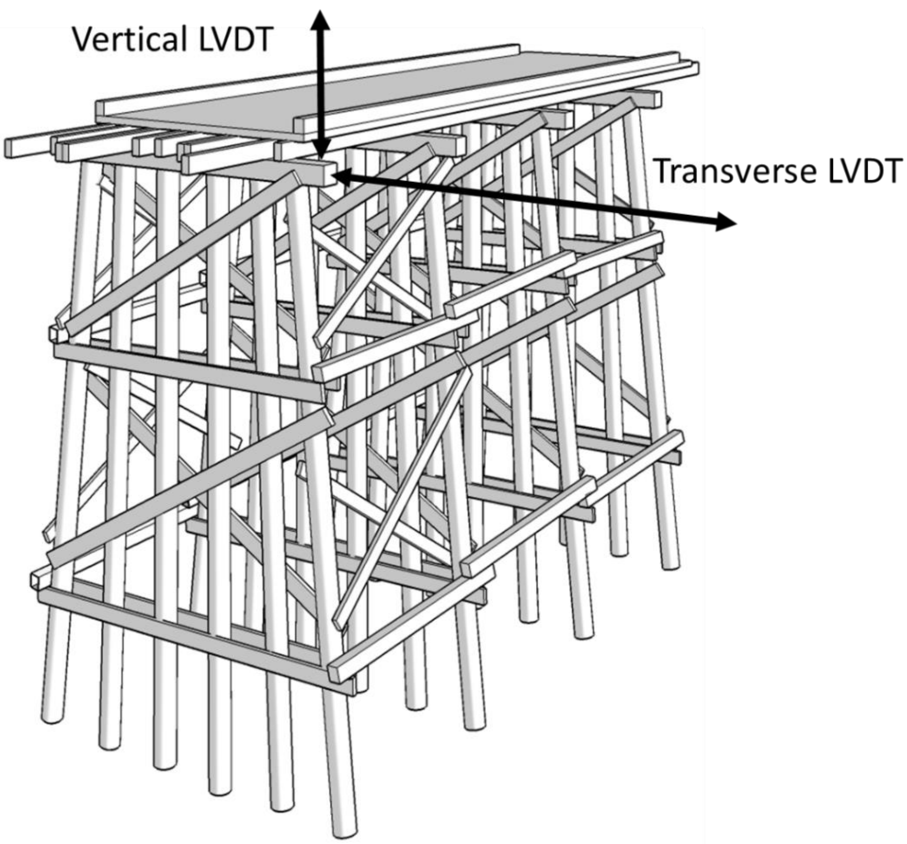


Figure 5.3. Relative location of displacement measurements in relation to the timber railroad bridge structure (partial view).

5.1.3 Test description

The same train car configuration/orientation crossed the bridge in both the South Bound (SB) and North Bound (NB) directions – five times each way, with speeds ranging from 8 km/h (5 mph) to 40 km/h (25 mph) in 8 km/h (5 mph) increments. The speeds of the trains were provided by the railroad both during the work train experiments and also under revenue service traffic crossing events. The authors confirmed the different speeds by using the sampling rate (100 Hz) during post-processing of the data, in conjunction with the geometry of the engines (also provided by the railroad). The SB WT test consisted of the locomotive pulling five cars with a total length of 110 m (360 ft); the distance between the first and last axel was 104 m (340 ft) (see Figure 5.4). The NB WT test consisted of the locomotive pushing the five loaded tank cars. Weights of the locomotive and cars were, respectively, 1112 kN (250 kip), and 1032 (232 kip), 1001 (225 kip), 1032 (232 kip), 1054 (237 kip), and 1023 kN (230 kip). Figure 4 shows the equivalent vertical loads applied onto the bridge. Because the second and third loads are adjacent to each other, they will be grouped as one. The direction of the WT was alternated, so no two consecutive tests crossed the bridge in the same direction. Bridge responses under four regular trains from revenue service traffic were also measured.

Both SB and NB responses were analyzed independently, because: (a) SB and NB train loading sequences were opposite of one another (see Figure 5.1(a)); (b) longitudinal forces (LF) in each case loaded the bridge in opposite directions; and (c) the bridge configuration (including boundary conditions) was not symmetric on either side of the pile bent.

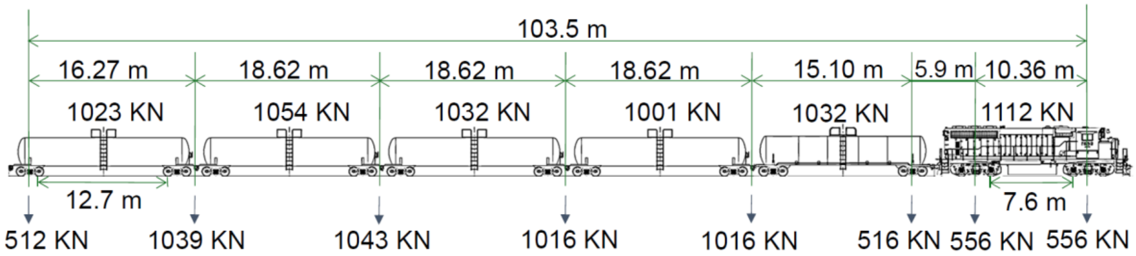


Figure 5.4 WT eight loading groups per influence region under axles of one engine and five cars.

5.2 Results: timber trestle displacements

5.2.1 Analysis of vertical displacements of pile caps

Vertical displacements did not change significantly under WTs running at different speeds. Table 5.1 shows a summary and statistical analysis of the vertical displacement data collected under all WTs. Precise train speeds were calculated by dividing the distance between the locomotive trucks (10.4 m (34 ft), per Figure 5.1) by the time elapsed between the two corresponding vertical responses. The statistical properties of the displacements were collected when any of the WT loads were within the pile influence region (see Figure 5.1(b)), defined as the portion of the bridge that transfers vertical loads to the pile bent (7.3 m (24 ft) for this particular pile bent). The first three columns are the mean, root mean square (RMS), and standard deviation (std) of bridge displacement for each train speed and direction. The fourth column is the maximum absolute peak-to-peak displacement (max), defined as the difference between the maximum and minimum displacements over time for a given WT. Statistical results of vertical displacements under WTs running in the considered speeds were similar.

As shown in Figure 5.5, the vertical displacements under both the slowest and fastest WTs running in the NB direction were almost identical. Moreover, all vertical amplitudes were small, which is consistent with trains with no significant wheel defects crossing the bridge at moderate speeds. To compare time histories under WTs running at different speeds, the horizontal axis (time) of the slowest WT was scaled down by the ratio of the two speeds. This comparison shows that the dynamic components are negligible at these speeds, and that there is no vertical response when all of the WT cars are outside of the influence area. Total displacement under each load was measured by computing the total distance between two consecutive local peaks in a displacement time history (including positive and negative). The vertical displacements included: (a) elastic shortening in the timber, estimated at 0.7 to 1.4 mm (0.03 to 0.06 in) (depending on the loading car); and (b) relative displacement of the pile into the soil. Based on (a) train car data, (b) total displacements, and (c) estimated shortening under each load, the relative pile displacement into the soil seems to be independent of vertical load during this test and occurs as soon as any significant load goes on the trestle. The elastic shortening of piles is not expected to change substantially over time, whereas the magnitude of relative displacement of the piles into the soil could indicate changes in bridge condition over time. As discussed and shown in Figure 5.4, seven loading events are considered for each WT, and the total displacements under each loading event are shown in Figure 5.6, including their averaged value and also the maximum value (under any car) for each speed level under NB WTs. Vertical displacements under each axle load do not change appreciably with train speed in this experiment.

Table 5.1. Summary of vertical displacements under WT running in two different directions and at five different speeds.

SB vertical displacements					**NB vertical displacements**				
speed (km/h)	**mean (mm)**	**RMS (mm)**	**std (mm)**	**max (mm)**	**speed (km/h)**	**mean (mm)**	**RMS (mm)**	**std (mm)**	**max (mm)**
8.7	-1.34	1.70	1.05	3.20	8.7	-1.31	1.69	1.06	3.41
16.2	-1.38	1.74	1.06	3.19	17.8	-1.29	1.67	1.06	3.29
23.3	-1.33	1.70	1.05	3.39	24.9	-1.28	1.65	1.04	3.24
33.9	-1.36	1.72	1.06	3.49	31.1	-1.22	1.61	1.06	3.38
41.5	-1.30	1.67	1.05	3.30	41.0	-1.27	1.67	1.08	3.57

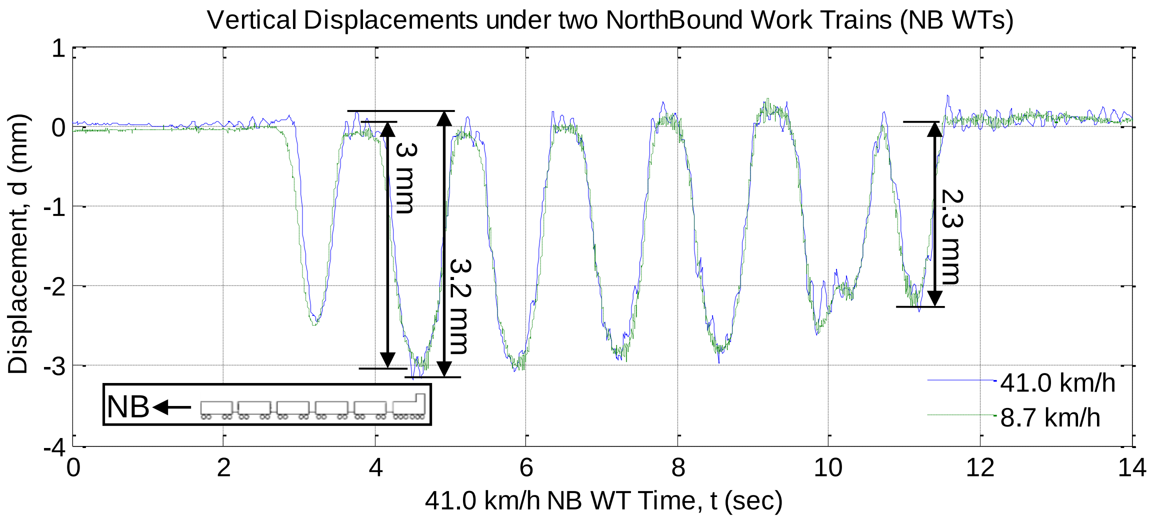


Figure 5.5. Similar vertical displacements under the seven loading events caused under axles of one engine and five cars running at two different speeds in the NB direction.

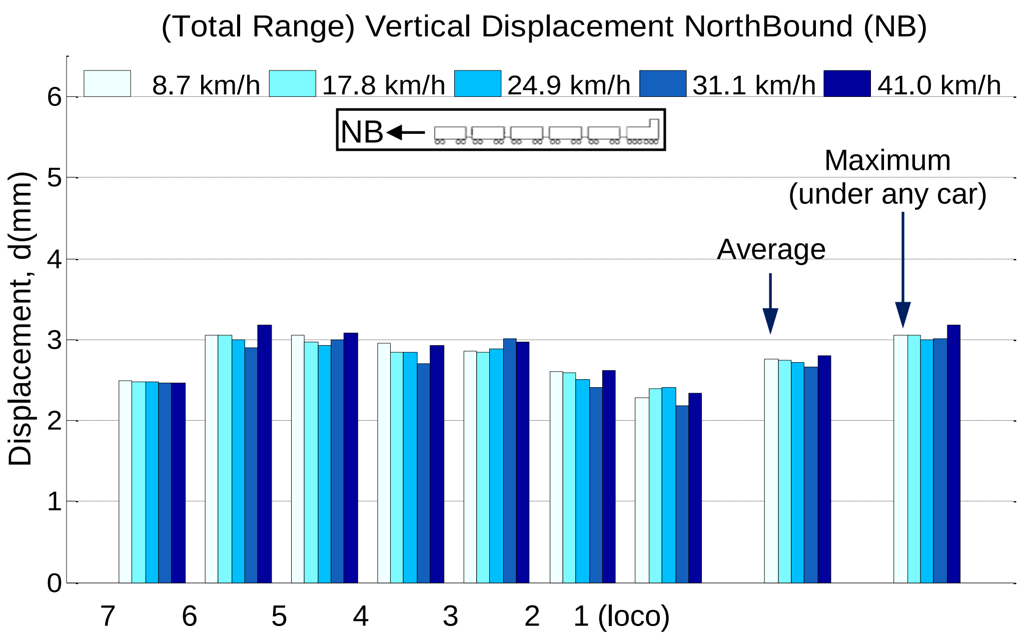


Figure 5.6. Total vertical displacement range vs. input loads for NB WTs.

Time histories of vertical displacement under WTs running in opposite directions had similar properties. To compare displacements, the response under the WT running in the NB direction was transposed (mirrored), and the horizontal axis (time) was scaled. Figure 5.7 shows the time history of vertical displacements under the SB WT running at 33.9 km/h (20.9 mph), as well as the mirrored (and scaled) time history under the NB WT running at 31.1 km/h (19.1 mph). Analyses of vertical displacements for loading events under SB WTs are identical than those under NB WTs. Time histories of vertical displacements under WTs running in opposite directions were similar for each of the speed levels of this experiment.

The vertical displacements are affected little by the speed and direction of motion of the train.

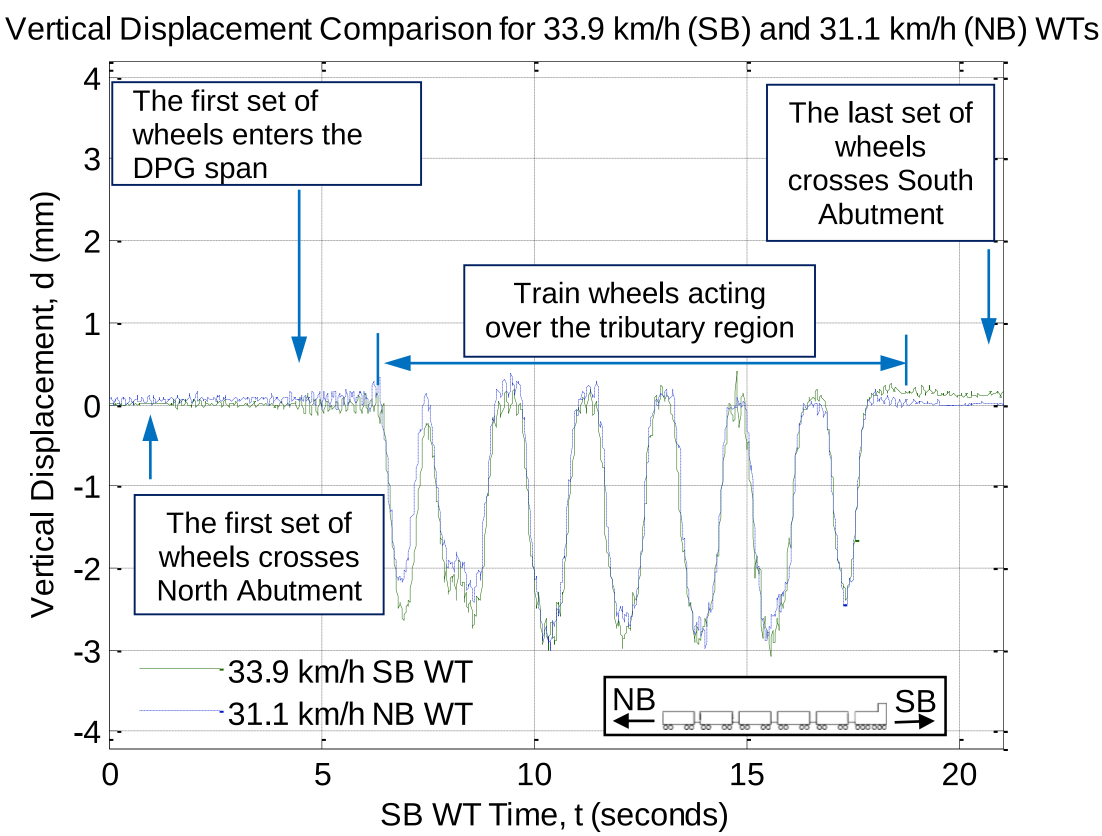


Figure 5.7. Vertical displacement comparison for 33.9 km/h (SB) and 31.1 km/h (NB) WT.

5.2.2 Analysis of transverse displacements of pile caps

Transverse displacements, on the other hand, did change significantly under WTs running at different speeds/directions. Table 5.2 shows a summary and statistical analysis of the transverse displacement data collected under all WTs. The maximum absolute peak-to-peak displacement (defined as the maximum displacement minus the minimum displacement for the total duration of the crossing event), 7.39 mm (0.29 in), occurred in the transverse direction under the NB WT running at 31.1 km/h, and not at the maximum train speed of 41.0 km/h (25.2 mph), for which the displacement was 6.67 mm. This result was not necessarily expected because reducing the operating speed is a method often used to increase the overall safety of the operations.

Table 5.2. Summary of transverse displacements under 10 WTs.

SB transverse displacements					**NB transverse displacements**				
speed (km/h)	**mean (mm)**	**RMS (mm)**	**std (mm)**	**max (mm)**	**speed (km/h)**	**mean (mm)**	**RMS (mm)**	**std (mm)**	**max (mm)**
8.7	-0.70	0.80	0.38	1.99	8.7	-0.46	0.61	0.40	2.32
16.2	-0.75	0.87	0.43	2.45	17.8	-0.46	0.69	0.52	3.12
23.3	-0.51	0.75	0.55	3.07	24.9	-0.57	1.10	0.94	5.31
33.9	-0.55	0.83	0.62	3.31	31.1	-0.56	1.20	1.06	7.39
41.5	-0.82	1.25	0.94	4.60	41.0	-0.58	1.16	1.00	6.67

Transverse displacements are different under WTs running in opposite directions (Table 5.2), primarily due to the asymmetry of the bridge (Figure 1). Under SB WTs, all four statistical properties had the highest value under the WT running at 41.5 km/h (25.5 mph). With the exception of the mean and RMS under SB WTs running at 23.3 and 33.9 km/h (14.4 and 20.9 mph), the statistical properties of displacements increased for SB WTs running at faster speeds. Under NB WTs, all RMSs, standard deviations, and maximum ranges for the WT running at 31.1 km/h (19.1 mph) were higher than values under WTs running at any other speed. With the exception of the means, the statistical properties of displacements have higher values for the NB WT running at 31.1 km/h (19.1 mph). The properties of the transverse displacements are different under WTs running at the considered speed and direction.

Transverse displacements are nonlinear with respect to speed and direction, and furthermore maximum transverse displacements occur under different cars when the WT is crossing in opposite directions. Figure 5.8 shows time histories of transverse displacement under the SB WT running at 33.9 km/h (20.9 mph) and for the NB WT running at 31.1 km/h (19.1 mph). To plot them together, the time of the SB WT was the taken as a reference, while the corresponding transverse displacement of the NB WT was mirrored and scaled. Maximum positive and negative displacements are clearly different for different cars and directions. For SB WTs, the average and maximum displacements under each car increased with train speed (Figure 5.9). Under NB WTs, the average and maximum displacements are significantly higher for the three higher speeds than for the prior two (Figure 5.10). In a situation where train speeds were lowered from 41.0 km/h (25.2 mph) to reduce lateral displacements, larger displacements could in fact be generated instead, which is somewhat counterintuitive and needs further explanation. Additional related work from Moreu et al. (2012b) also estimated displacements at two different pile bents, indirectly using the accelerometer data. Reference-free accelerometers proved to be effective for simple displacement estimation of transverse displacements under traffic (Moreu et al. 2012a). Results of that work indicated that the two pile bents immediately to the North of this one also showed larger transverse displacements under NB WTs running at 31.1 km/h (19.1 mph) than under NB WTs running at 41.0 km/h (25.2 mph).

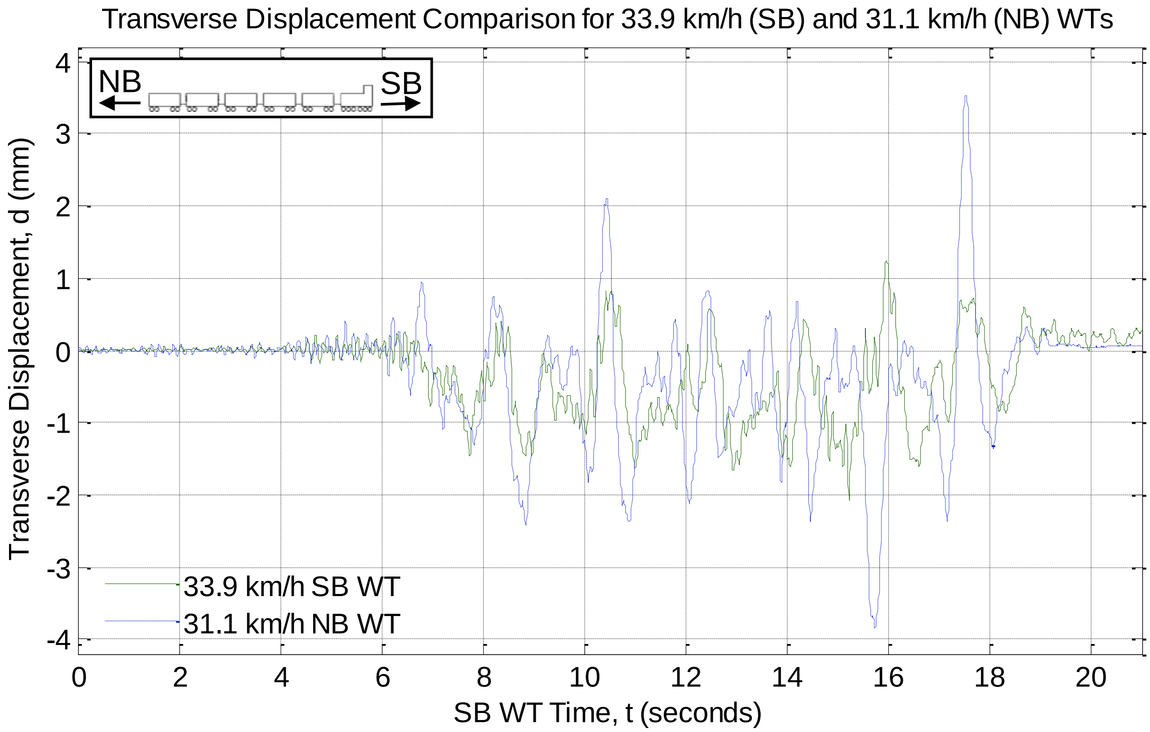


Figure 5.8. Comparison of transverse displacement under both SB and NB WTs.

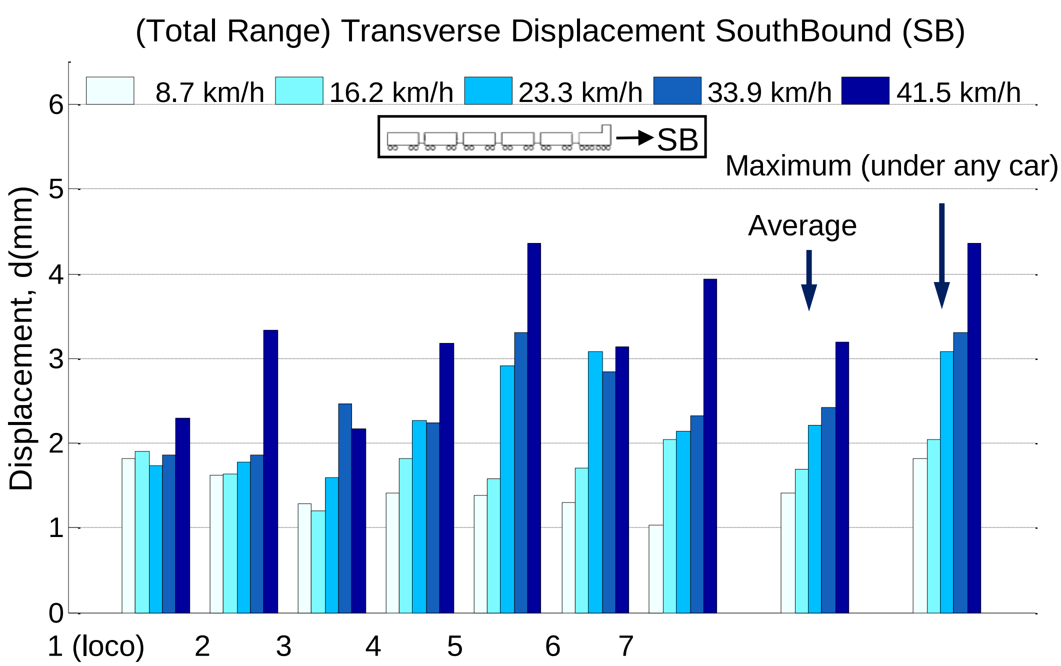


Figure 5.9. Total transverse displacement range vs. input loads for SB WTs.

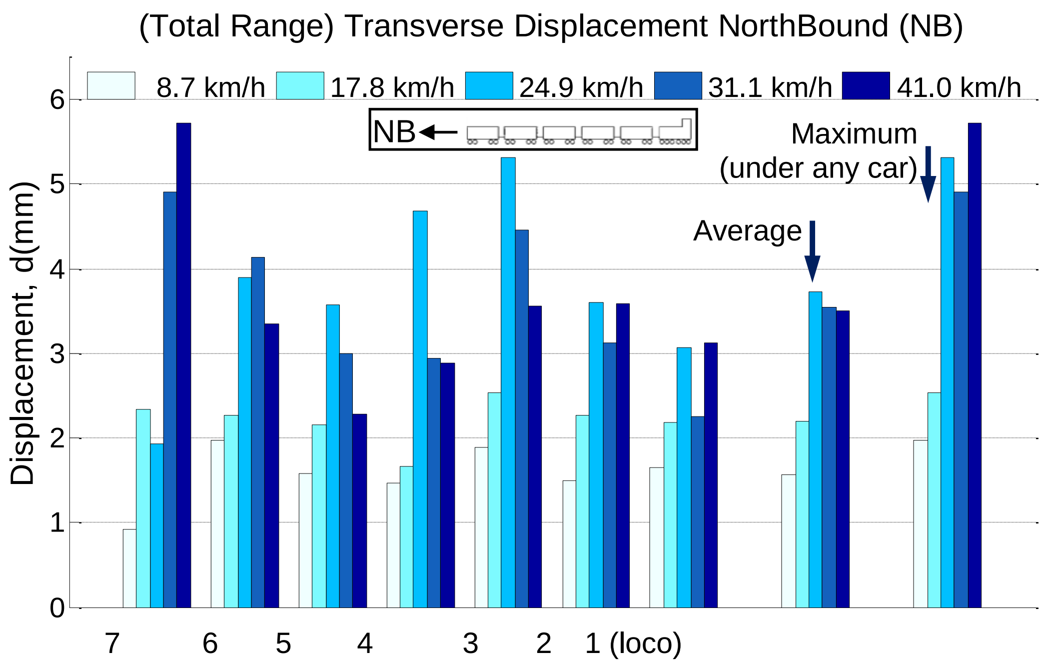


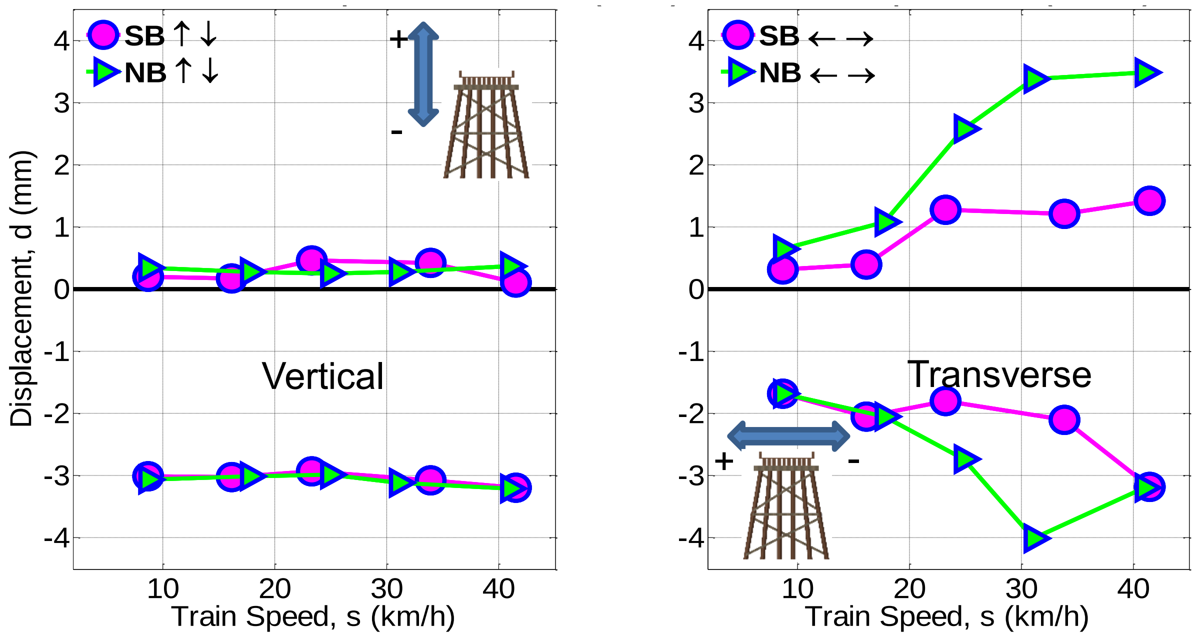
Figure 5.10. Total transverse displacement range vs. input loads for NB WTs.

5.2.3 Comparison of vertical and transverse displacements

Transverse displacements change under different train speeds and directions and can be used as indicators to measure the impact of speed to the safety of railroad operations. Figure 5.11 shows the extreme displacements in both the (a) vertical and (b) transverse direction versus train speed. Each mark represents a maximum or minimum displacement under the WT. Both design and maintenance regulations typically limit vertical responses at mid-span, whereas in this particular case transverse displacements can better identify changes in bridge performance, such as the higher displacements under a NB WT running at 31.1 km/h (19.1 mph). The track, speed, and loading properties provide some evidence that harmonic roll could be causing higher displacements at this speed in the NB direction. This phenomenon has the potential to create unsafe operations. Reducing the operating speed is a method often used to increase the overall safety of the operations. Transverse displacements change under trains running at different speeds/directions; therefore, transverse displacement changes may be a factor to assist controlling railroad operations safety under different loading conditions and/or over time.

5.3 Frequency and harmonic roll

Analysis of displacement data in the frequency domain (Figure 5.12) finds that harmonic roll may well be the cause for larger transverse displacements under NB WTs running at lower speeds. Tank cars tend to have a higher center of gravity compared to most other rail cars. The higher center of gravity is a contributing factor in vehicle car body harmonic roll behavior. Only frequencies under 5 Hz are shown because, according to the literature review, this bandwidth is sufficient for harmonic roll analysis. The frequency analysis of vertical response shows frequencies caused by WT axles crossing the joints at each considered speed level (0.18, 0.27, 0.45, 0.54, and 0.72 Hz). The frequency analysis of transverse response shows: (a) a dominant frequency (of 1.26 Hz) under the NB WT at 24.9 km/h (15.3 mph); (b) harmonic roll (at 0.81 Hz) under the NB WT at 31.1 km/h (19.1 mph), plus an additional dominant frequency of 1.35 Hz; and (c) a dominant frequency of 1.35 Hz under the NB WT running at 41.0 km/h (25.2 mph). In general, the frequency analyses of the two directions indicate frequencies excited by NB WTs at speeds that are in general associated with harmonic roll.



(a) (b)

Figure 5.11. Maximum and minimum displacements, d (mm) vs. train speed, s (km/h) for (a) vertical and (b) transverse directions.

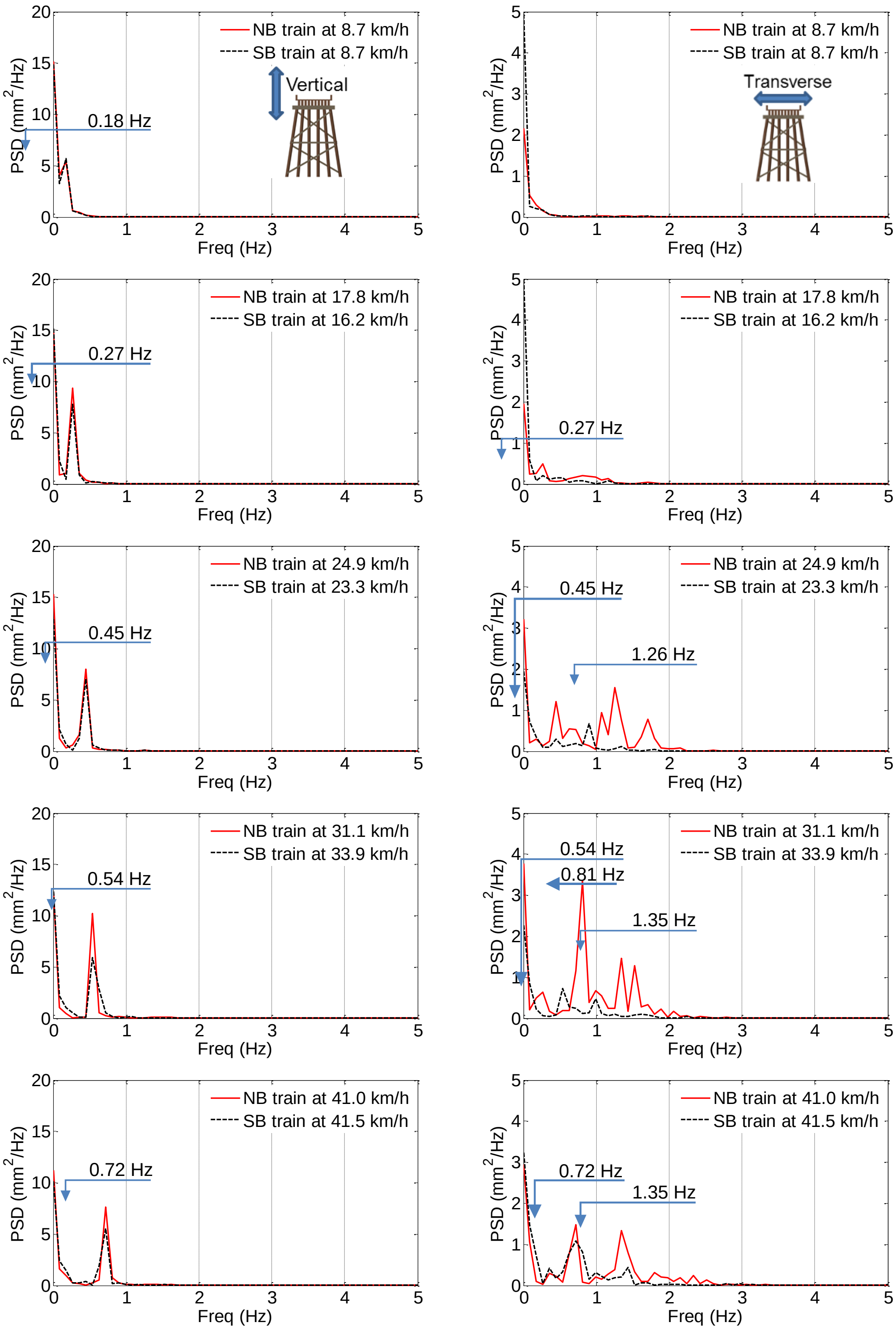
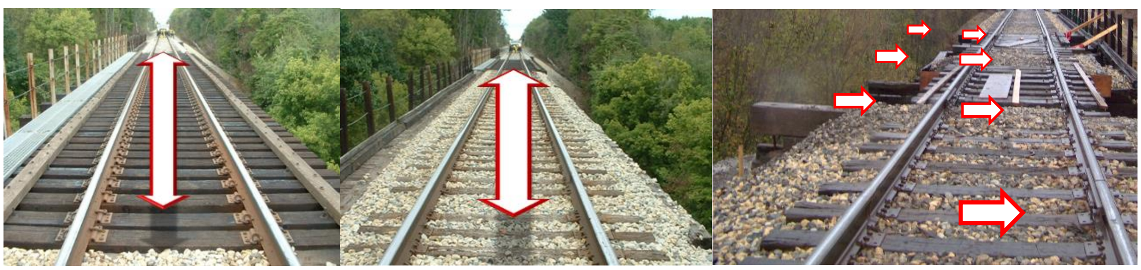


Figure 5.12. Frequency response to WTs in vertical (left) and transverse (right) direction

The apparent harmonic roll under NB WTs is likely due to the asymmetry in bridge and track properties on each side of the pile bent being studied. SB WTs crossed over the DPG before the pile bent (Figure 5.1); NB WTs traveled toward the sensors from the Southern timber trestle approach. The DPG main span track on the North side had an open deck with timber ties of 20 cm x 25 cm (8 in. x 10 in.) spaced every 30 cm (1 ft) (Figure 5.13(a)), and the rail was continuous. The trestle approach, on the other hand, had timber ties of 23 cm x 18 cm (7 in. x 9 in.) spaced every 54 cm (21 in.) (Figure 5.13(b) and (c)) on ballast, with jointed track. NB trains run on the jointed track of the Southern timber trestle approach before reaching the pile bent, building up harmonic roll under specific speeds and directions, as shown in the analysis of displacements. NB WT cars moved more than SB WT cars because they rode on jointed track prior to the pile bent.



(a) (b) (c)

Figure 5.13. Bridge at track level: (a) new open deck track at main span, (b) ballasted deck at South approach, and (c) jointed track detail in trestle approach.

The “rock-and-roll” of loaded tank cars would explain why the railroad sought to limit the train speed on this bridge. Larger displacements under the NB WT running at 31.1 km/h (19.1 mph) than for the NB WT running at 41.0 km/h (25.2 mph) captured the harmonic roll effect of the loaded cars crossing the bridge over jointed track in the NB direction. The railroad had to reduce the traffic speeds twice, from 97 km/h (60 mph) and 64 km/h (40 mph), (equivalent to track classes 4 and 3, respectively), and the cause may in part have been the amplification of displacements (resonance) at multiples of the harmonic speeds (and also related to frequencies between 1.26 and 1.35 Hz, found in the frequency analysis). The railroad reduced train speeds as part of the construction process, so there are not excessive deflections in this case. This analysis shows evidence that relates harmonic roll in this bridge to the prior two slow orders.

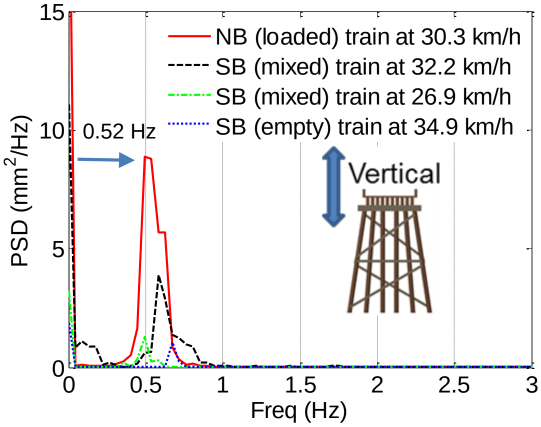
5.4 Displacements under revenue service traffic: analysis and results

Transverse displacements from regular train traffic can further quantify safety of railroad operations related to given speeds and directions. Table 5.3 shows a summary of maximum transverse displacements under revenue service traffic. To compare maximum displacements under different trains, the maximum displacement was obtained under (a) locomotives (column 4, locomotive), (b) any freight car within the train (column 5, car), and (c) the entire train crossing event (column 6, total). Maximum transverse displacement values under revenue service traffic and WTs were similar under similar traffic speeds and directions. Train speeds and vertical displacements under locomotive loadings were comparable for the four trains. For similar vertical loadings and speeds, maximum transverse displacements were higher under the NB train than the SB trains. Analysis of transverse displacements under regular traffic can identify which direction may be more critical for safety of railroad operations.

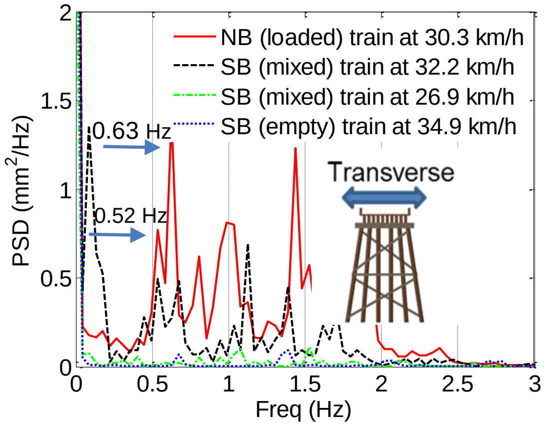
Table 5.3. Maximum transverse displacement amplitudes under revenue service traffic.

train properties			**maximum transverse displacement, d (mm)**		
traffic direction	**train speed, s (km/h)**	**load type**	**locomotive**	**car**	**total**
SB	34.9	empty	3.51	3.18	3.51
SB	26.9	mixed	3.16	4.09	4.78
SB	32.2	mixed	2.77	4.83	6.03
NB	30.3	coal	4.37	6.74	7.16

This analysis of revenue service traffic confirms that different railroad bridge boundary conditions (and track condition) affect railroad bridge response differently, and that the data under NB train had signs of harmonic roll. This result supports the idea that railroad bridge safety should consider dynamic effects, in addition to vehicle weight and speed. The maximum transverse displacement amplitude occurred as loaded train cars passed over the bent, and not under locomotives (which have similar axle weight). This result indicates that the vehicle-track-train interaction after multiple cars crossing over the trestle augments transverse displacements. Figure 5.14 shows frequency analysis under four trains for the vertical (a) and transverse (b) direction. The transverse response analysis indicates that the NB train running at 30.3 km/h (18.6 mph) excited the harmonic rolling effect at 0.63 Hz (and also higher), which is consistent with the frequency analysis under WTs. The vertical response analysis captured the frequencies generated by the speed of each train (0.49, 0.52, 0.58, and 0.63 Hz). The SB mixed train running at 32.2 km/h (19.8 mph) excited the bridge laterally with multiple frequencies under 2 Hz. SB trains running at 26.9 km/h (16.6 mph) and 34.9 km/h (24.5 mph) did not show evidence of harmonic roll, nor of comparable vibration levels for lower frequencies.



(a)



(b)

Figure 5.14. Frequency response to revenue service traffic: (a) vertical and (b) transverse direction.

5.5 Transverse displacements under different bridge conditions

Transverse displacements may increase with decay in bridge condition. Figure 5.15 shows the maximum transverse displacements of the Bluford bridge, as well as those of three other timber trestles under revenue service traffic. The three other bridges are named Fulton, Freeport, and Yazoo, and all are located in the Midwestern U.S. Measurements were made using a laser projected from a fixed point and a video camera that recorded relative transverse displacements of pile caps under trains. Pile heights for the Fulton, Freeport, and Yazoo bridges were 2.4, 2.4, and 6.4 m (8, 8, and 21 ft), respectively. The railroad knows which pile bents were more critical for each timber trestle based on their expert opinion and data from regular inspections. The various monitoring campaigns of both Fulton and Yazoo bridges occurred during construction activities affecting the structural condition of the bridge being replaced. These experiments did not record vertical displacements; as indicated previously, the vertical displacements were quite small and didn't appear to provide much information regarding the bridge condition. Different stages of construction at a given bridge are distinguished with different numbers, ranging from 1 (less construction) to 3 (more construction): a higher construction phase implies a larger percentage of the deck structure being removed (for the installation of new bridge), making it less stiff. Yazoo phases 1, 2, and 3 had 3, 3, and 5 deck panels, respectively, changed from ballast deck to open deck. The ultimate observed deflection was 41.3 mm (1 5/8 in.) under a loaded coal train, with both spans on either side of bent four converted to open deck. Albeit temporary, less stiff deck structure implies more critical structural condition.

For the four groups of measurements for the Yazoo bridge, transverse displacement levels of selected pile bents of 19.1 mm (3/4 in.) or higher are potentially related to railroad operations with railroad bridges during construction (Yazoo Phases 1, 2, and 3). For the three groups of data labeled Bluford, Freeport, and Yazoo (before advanced phases of construction), 92% of the maximum transverse displacements were under 12.7 mm (1/2 in.), 62% under 9.05 mm (3/8 in.), and 38% under 6.3 mm (1/4 in.). The top four maximum transverse displacements occurred at Yazoo phase 3 under freight trains. The data shows that the lower structural conditions cause higher transverse displacements. Because displacements larger than 25.4 mm (1 in.) occurred for all freight traffic at Yazoo Phase 3, transverse displacements exceeding 25.4 mm (1 in.) appear to indicate that the construction process has reduced the lateral stiffness of the bridge. The railroad installed additional bracing at Yazoo Phase 3 after field observations of trains and bridge movements to ensure the safety of rail operations before the bridge replacement. Based on the measurements for this bridge, the ratio between this transverse displacements to the height of the pile is proposed as a preliminary metric for possible action. In this case the height of the pile was 6.4 m (21ft) which leads to a one in 252 ratio.

The maximum transverse displacements under freight trains may possibly be caused by dynamic vehicle-track-bridge interactions and not solely based on the weight of the train cars. The maximum displacement under the coal train crossing Yazoo bridge in phase 3 was under car 55 (for a total of 86 cars). The reason that the transverse displacements occur under longer trains for higher phases of construction may be explained by harmonic roll being excited under repetitive large loads interacting with timber trestles, and/or additional dynamic excitations caused under a high number of large loads. The displacements under Amtrak trains were similar for the different phases (12.7 mm, 1/2 in.) because there is not repetitive interaction between heavy cars and the trestle (passenger trains are both lighter and shorter than freight trains, and their equipment has much better suspensions compared to freight equipment, for reasons of passenger comfort, as well as relatively constant vertical load). The maximum displacement under Amtrak trains was smaller than maximum displacements under freight trains; this difference was greater than would have been predicted based on linearly scaling with respect to the weights of their respective locomotives and cars.

Track alignment data collected by track geometry vehicles (consisting of a limited number of cars and measuring rail data to control safe operations) may not capture dynamic interaction of long and heavily loaded trains and trestles. Typical freight trains of over one hundred cars are expected to provide the maximum transverse displacements for timber railroad bridges. The current track alignment deviation limits for Class 2 and 3 tangent tracks (with freight trains limited to a maximum of 40 km/h (25 mph) and 60 km/h (40 mph)) are 75 and 44 mm (3 and 1¾ inches), respectively (FRA, 2014). The fact that this bridge was carrying traffic at these speed levels shows that even when the track could be performing under satisfactory safety levels, the changes of displacements in the bridge could show evidence of changes in bridge condition that would not be captured under current geometry track limits.

Transverse displacement is one factor that should be taken into consideration when establishing maintenance and replacement priorities, and the authors of this research suggest that it would be related to the height of the pile (*H*). Because the height (H) of the pile is 6.4 m (21 ft), a transverse displacement of $H/250$ is suggested as a possible point where overall bridge behavior under load should be further investigated by the bridge owner. Lower and upper limits should also be considered based on the heights of the population of trestles in North America. The transverse displacement of the bridge may possibly provide information about the condition of the bridge. Based on transverse displacements, railroads can make decisions based on objective information. In the Yazoo bridge, displacements over 24.5 mm (1 mm) are indication of excessive displacements that in general operations would require a slow order in this bridge and possibly additional maintenance and repair orders. Because this bridge was already scheduled for replacement in the near time, there was no need to order urgent replacement. The bridge was already under two slow orders prior to the monitoring. The measurement of excessive displacements ($d>H/250$) for a bridge under revenue service traffic and regular operations could possibly mean a slow order and the investigation of the structure to determine if maintenance, repair, or replacement would need to be scheduled. The implications of this decisions and their cost for the network will be presented in one example in Chapter 7 of this dissertation.

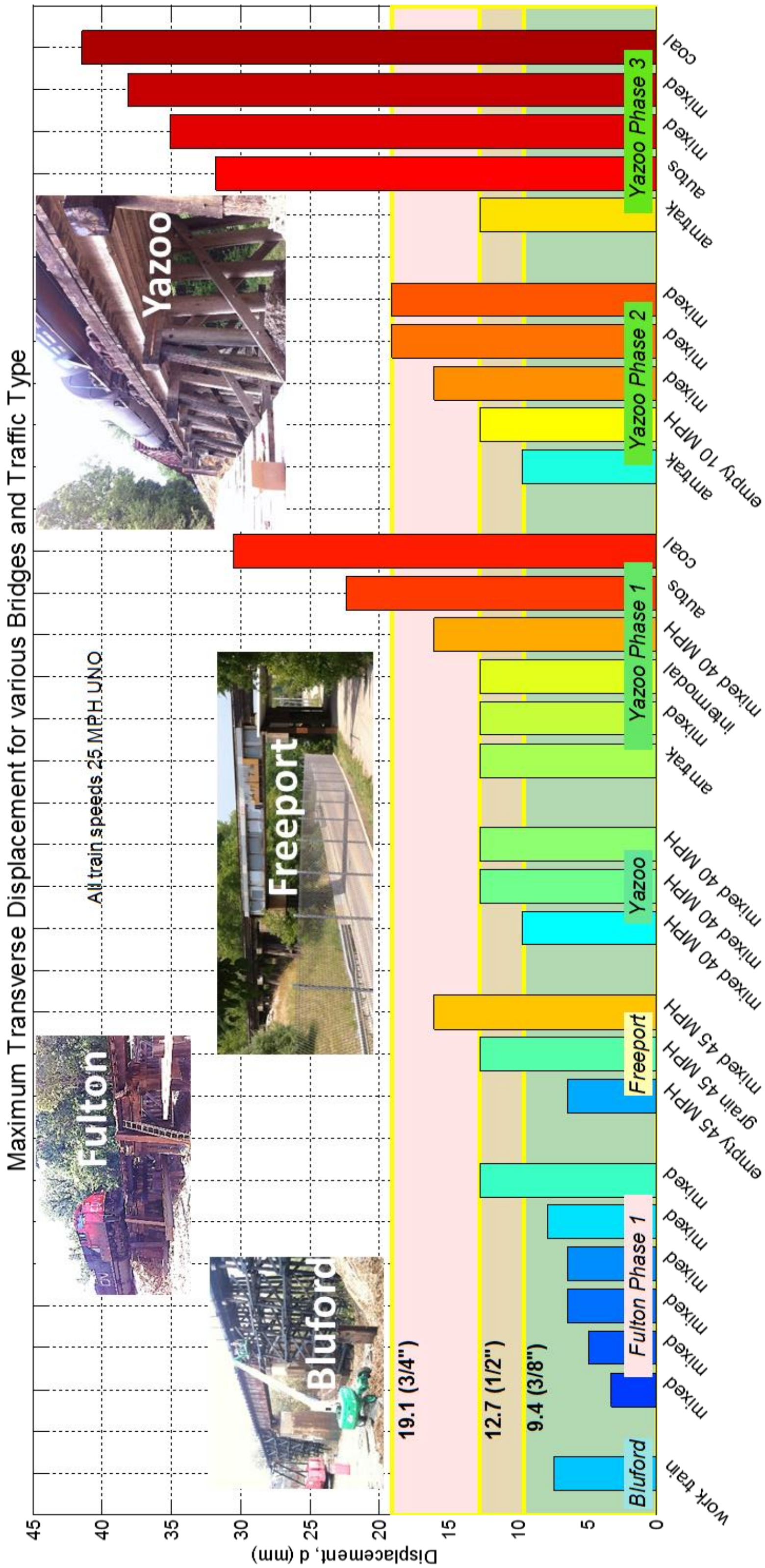


Figure 5.15. Maximum transverse displacements at four different bridges under different open-traffic conditions.

5.6 Conclusions

A recent survey identified that measuring both vertical and transverse displacements of bridges under trains is a top research need today. This paper summarizes analysis of vertical and transverse displacements of a Class I timber trestle under train loadings measured in the field. Vertical displacements captured the pseudo-static response and did not change appreciably with train speeds considered or with the direction of train travel. Based on the analysis, transverse displacements were apparently affected by speed, direction, and vehicle-track-bridge interaction under long heavy coal trains. In general, transverse absolute displacements increased with train speed.

Transverse displacement measurements captured evidence of harmonic roll under NB WTs running at 31.1 km/h (19.1 mph). This phenomenom was not found under WTs running in the SB direction because of different track conditions and boundary conditions at the bridge. According to this experiment, transverse displacements can capture less favorable timber trestle responses under traffic due to dynamic vehicle-track-bridge interaction between loaded and long trains. However, further research should be conducted in similar timber railroad bridge types and locations with detailed monitoring. Based on 29 measurements for 4 timber trestle railroad bridges, the authors found that transverse displacements exceeding 25.4 mm (1 inch) are likely to indicate deteriorated (or abnormal) bridge conditions. The authors suggest that transverse displacements exceeding H/250 indicate the need for further investigations when establishing maintenance and replacement priorities. Finally, these results indicate that railroad managers may be able to use maximum transverse displacements as one of the measures to prioritize further inspections, maintenance or bridge replacements.

CHAPTER 6 REFERENCE-FREE DISPLACEMENT ESTIMATION FOR A RAILROAD BRIDGE ASSESSMENT USING WIRELESS SMART SENSORS

This chapter proposes and investigates assessing railroad bridge condition from displacements determined using reference-free estimations from accelerations collected with Wireless Smart Sensors (WSS). Actual displacements measured from a timber bridge trestle pile bent have been compared with estimated displacements under different traffic conditions; see Figure 6.1. Estimated transverse displacements from multiple pile bents assisted in assessing bridge condition at different locations. Results for the estimation of vertical and longitudinal displacements identified additional work required to fully estimate non-zero mean displacement with multi-metric sensing, based on results from this study. This chapter validates using WSS for estimating transverse displacements under open traffic to provide inexpensive, effective, and simplified campaign monitoring of railroad bridges.

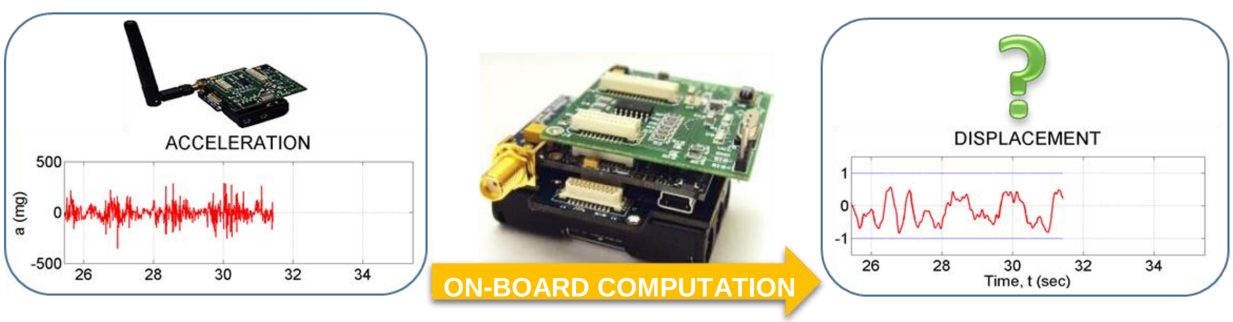


Figure 6.1. Reference-free displacement estimation using wireless smart sensors (WSS)

6.1 Background

This section provides background regarding the method employed in this research to estimate displacements from measured accelerations. To eliminate the need for information about double integration and unknown constants of integration, Lee et al. (2010) proposed minimizing the difference between the double derivative of the displacement and the acceleration within a finite time interval. The objective function to be minimized can be written as:

$$\min_{\mathrm{u}} \Pi = \frac{1}{2}\|\mathbf{L}\mathbf{u} - (\Delta \mathrm{t})^2 \mathbf{L}_{\mathrm{a}}\bar{\mathbf{a}}\|_2^2 + \frac{\lambda^2}{2}\|\mathbf{u}\|_2^2 \tag{6.1}$$

, where u, Δt, $\bar{\mathbf{a}}$, $\mathbf{L}_{\mathrm{a}}$, L, $|\cdot|_2$, and λ, are estimated displacement, time increment, measured acceleration, integrator operator and diagonal weighting matrix, 2-norm of a vector, and optimal regularization factor, respectively.

The optimal regularization factor λ is presented in equation (6.2), and it depends on the number of data in the time window (*N*):

$$\lambda = 46.81 \cdot N^{-1.95} \tag{6.2}$$

The size of the time window is usually two or three times the longest estimated period of the target structure. Using the measured acceleration and equation (6.5), the estimated displacement (u) is:

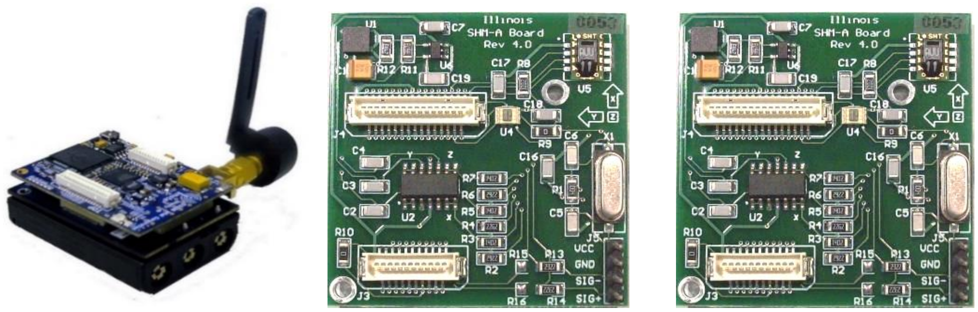
$$\mathbf{u} = (\mathbf{L}^{\mathrm{T}}\mathbf{L} + \lambda^2\mathbf{I})^{-1}\mathrm{L}^{\mathrm{T}}\mathbf{L}_{\mathrm{a}}\bar{\mathbf{a}}(\Delta t)^2 = \mathbf{C}\bar{\mathbf{a}}(\Delta t)^2 \tag{6.3}$$

, where I is the identity matrix and C becomes the coefficient matrix for the displacement reconstruction.

Park et al. (2011b; 2014) embedded this algorithm in WSS and conducted laboratory tests to demonstrate the potential of this approach, which they called *Independent processing-based Displacement Estimation using Acceleration (IDEA)*. With the displacement estimation algorithm programmed, the WSS network performs decentralized independent processing to estimate displacements at each sensor location (Park et al. 2013a). The validity of the proposed method was experimentally demonstrated on a three-story shear building under free vibration. The method was improved using multimetric approaches that also use strain to estimate displacements of highway bridges (Park et al. 2013b; Park et al. 2014). This algorithm is employed herein for direct estimation of railroad bridge deflections from accelerations measured under live train load.

6.2 Wireless Smart Sensors

The Imote2, a platform for WSS developed by Intel (see Figure 6.2(a); ISHMP 2014) was used to monitor this bridge. The Imote2 includes a high-performance X-scale processor (PXA27x), permitting speed adjustments, based on application demands and power management, ranging from 13MHz to 416MHz. The Imote2 has 256K SRAM, 32MB FLASH, and 32MB SDRAM, which enables the intense onboard calculations required for SHM applications, as well as storage of longer term measurements when needed. The University of Illinois developed sensor boards (ISHMP 2014) that can be stacked on the Imote2 via two connectors to facilitate sensing, including a general-purpose accelerometer board (SHM-A) (Rice et al. 2009), see Figure 6.2(b). Rice and Spencer (2008) validated the accuracy of the SHM-A board using a capacitive accelerometer (PCB Model 3701G3FA3G) (Piezotronics, 2007). Because their tri-axial accelerometers have a very low noise and are inexpensive, such system provides an effective tool to measure bridge responses under train crossing events. Services such as drivers and software were available from the ISHMP Service Toolsuite, which allows choosing monitoring parameters, such as the sampling rate and filtering. For this project, accelerations were collected using a sampling rate of 280 Hz and digital low-pass filtering with a cut-off frequency of 70 Hz.



(a) (b)

Figure 6.2. (a) Imote2 sensor board with antenna and stacked on battery board, (b) SHM-A sensor board (above and below views) (ISHMP 2014).

Figure 6.3(a) shows the complete sensor assemblage, while Figure 6.3(b) shows the final storage inside a campaign monitoring enclosure ready for field monitoring. The material of the pile cap was reinforced concrete and the enclosure has magnetic supports. Therefore, alternative installation of the WSS on both the bent cap and the scaffolding was performed in a few hours prior to monitoring. Before sensor installation on the bridge, different attachment tests were conducted, which indicated that the most efficient way to attach WSS to concrete was by epoxying and anchor-bolting a ¼ in. steel plate to the bent cap; this plate then became a base for the magnets of the sensor enclosures.

Figure 6.3. (a) Stacked WSS board with battery board and SHM-A sensor board, (b) enclosure for campaign monitoring applications (ISHMP 2014).

WSS were installed and removed from the railroad bridge with less time, cost, and effort than LVDTs. The total mass added to the pile cap was very small relative to the mass of the pile cap, and the effect of localized vibrations to the results were assumed negligible for this experiment. The installation of the supporting plate to attach the WSS at the bridge took 30 minutes by one person using a men lift available at the site (in other bridge scenarios without ground access to the bridge, inspectors could attach WSS to the bridge pile caps from the track level using a regular cherry picker crane on rail). The three WSS were controlled wirelessly by a personal laptop PC, so that one bridge inspector could install and operate the WSS system autonomously. Scaffolding design required negotiation and approvals. Machinery and construction personnel were already mobilized at this bridge for construction operations, however erection took two days and was expensive. A special bridge testing vehicle moveable laboratory was parked by the bridge to install the acquisition system for the two LVDTs, employing one day (each) for mobilization and demobilization due to the height of the piles. Because WSS are easier to install, operate, and their cost is very low, they are convenient tools to monitor displacements under revenue service traffic.

6.3 Displacement estimations

Figure 6.4 shows the relative location of the LVDTs and accelerometers, and their orientations, with respect to a partial 3D view of the timber trestle. This research uses reference-free accelerations to estimate the displacements of the pile bents. The stringer's condition at the time of the experiment was not of concern, so they were not instrumented during the testing. The same experiments conducted in Chapter 4 of this dissertation were conducted to estimate reference-free displacements using WSS.

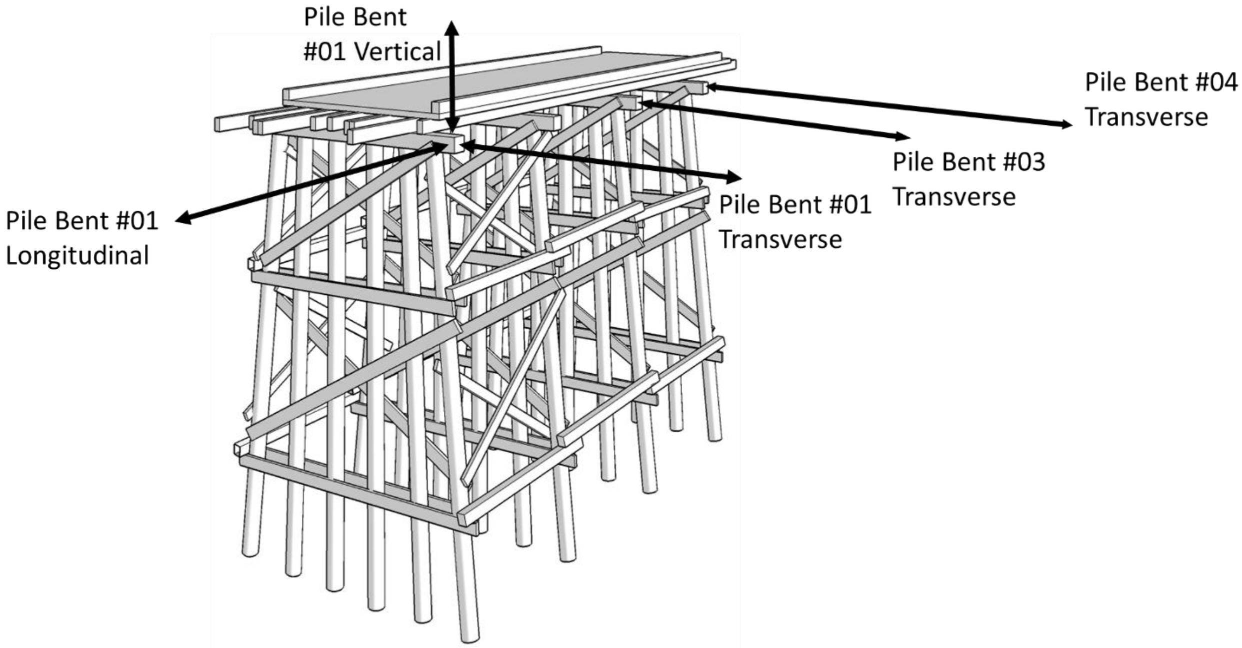


Figure 6.4. Location of displacement and acceleration measurements in relation to the timber railroad bridge structure and pile bents (partial view)

6.3.1 Transverse displacement estimations for pile bent 1

This section describes the proof of concept of comparisons between reference-free and traditional displacement measurements. WSS reference-free estimated transverse displacements and LVDT measured (actual) transverse displacements are similar in the time domain. Figure 6.5 shows the estimation of transverse displacements using accelerations under a WT running in the NB direction at 41 km/h (25 mph). The WSS displacements are labeled as “estimated” because they are computed using WSS, whereas the LVDT displacements are labeled as “measured” because they have actually been measured off the reference scaffolding. The result of this comparison shows that even when the estimated displacement captures the main features of the measured displacement (generally similar amplitudes and phasing), the estimated amplitudes are slightly smaller than the measured amplitudes. For all of the live load tests the comparison between reference-free and traditional displacement measurements in the time domain are similar.

Measured displacements have a non-zero mean component that is not present in the estimated displacements. Underestimating bridge response can be avoided by quantifying the error range so that the estimations can be corrected, by amplifying the reference-free measurements with a constant value that conservatively corrects the disagreement between the two measurements. This research quantifies the underestimation of WSS reference-free estimated displacements in order to provide information for bridge assessment that can be more accurate than current practice standards. The following parts of this section further analyze in detail the comparison between measured and estimated displacements, in order to quantify the error and inform corrective methods.

Although WSS cannot estimate the total displacement accurately, and the values need to be corrected based on a correction factor, WSS can precisely estimate the maximum total dynamic displacement and show changes of displacements under WT at different directions and speeds. This reinforces the interest in using WSS for bridge inspection, because some railroads are interested in the total dynamic movement under a train (sum of absolute dynamic displacement in both the negative and positive directions). If the pseudo-static component is small in comparison to the total displacement, the dynamic displacement will capture the majority of the bridge displacement under trains. LVDT displacements (measured) have a pseudo-static component caused by the weight of the car on the bridge. To obtain the dynamic response of the bridge from the LVDT measurements, the pseudo-static component of the LVDT displacements is filtered (or de-trended). Although some errors in WSS total displacement values were identified when compared with LVDT measurements, estimated WSS dynamic displacement matches well to the measured LVDT dynamic component of the transverse responses under WTs at different speeds and directions. The following figures compare dynamic displacements of WSS and LVDT measurements.

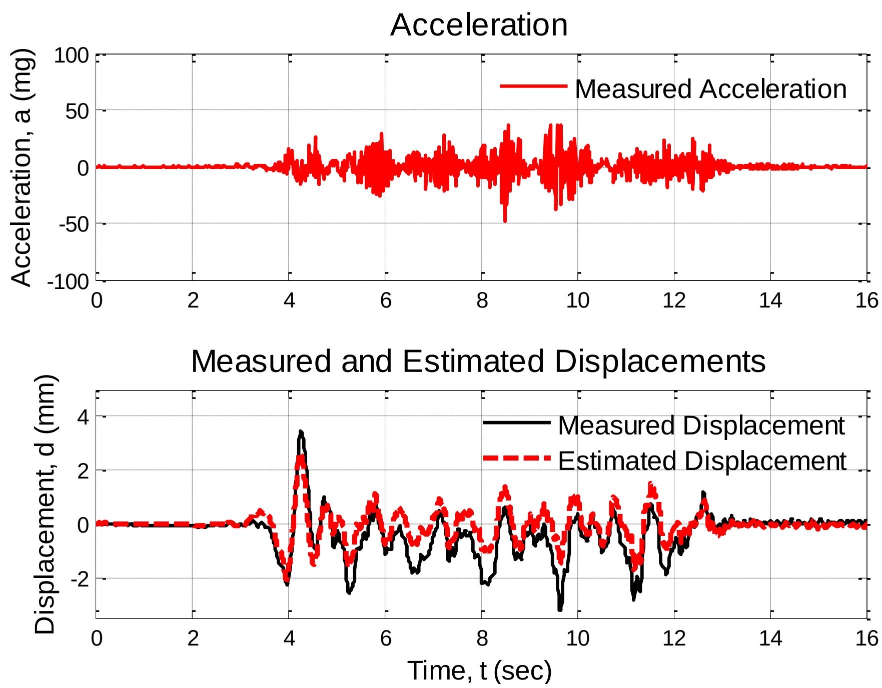


Figure 6.5. Estimation of transverse displacements using accelerations for pile bent 1 under NB WT at 41.0 km/h

The data collected on the scaffolding demonstrated that the relative vibration of the pile bents in relation to the scaffolding (from where they were measured) had only a very small effect on the estimation. Using the accelerations measured by the Imote2 sensor c (Figure 5.2), the scaffolding estimated displacement could be subtracted from the bent cap estimated displacement, as shown in Figure 6.6. From the comparison between these estimated displacements, the scaffolding vibration had a negligible effect on the displacement estimation.

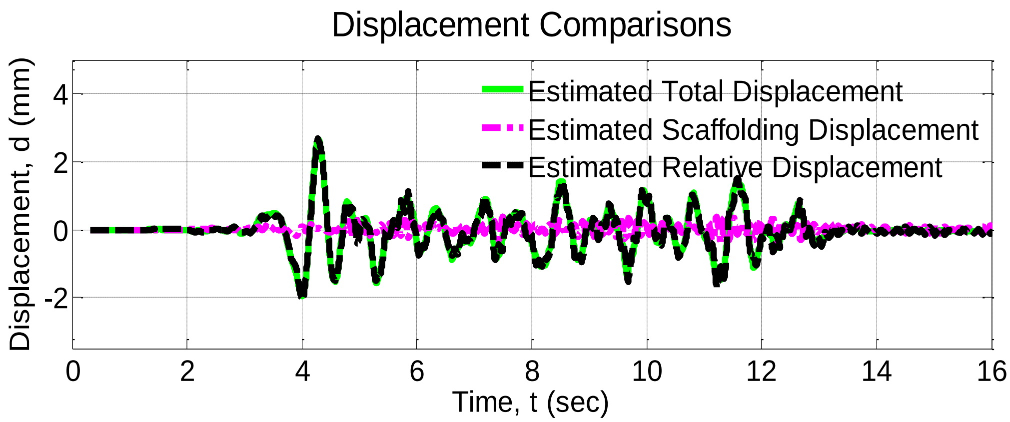


Figure 6.6. Estimated displacements of scaffolding under NB WT at 41.0 km/h

Figure 6.7 shows a complete comparison between WSS displacements and the de-trended measured LVDT displacements under the WT traveling at all the different speeds and directions. The data collected on the scaffolding demonstrated that the relative vibration of the scaffolding had only a very small effect on the pile bent displacement measurements. In general, the estimations of transverse displacements are very close to the measured de-trended displacements, independent of the speed and direction of the trains. Amplitudes of both measured and estimated displacements increase with the speed of WT, and are larger under NB WT than SB WT. Root mean square (RMS) values of the error and their percentage relative to LVDT peak displacement measurements are shown. WSS data was not recorded for live load tests for SB WT at 8.7 km/h (5.4 mi/h) and 16.2 km/h (10.1 mi/h). All recorded RMS error values are under 0.45 mm (1/64 in.) and 13.6%. The WSS reference-free displacements estimate LVDT displacements with small RMS errors.

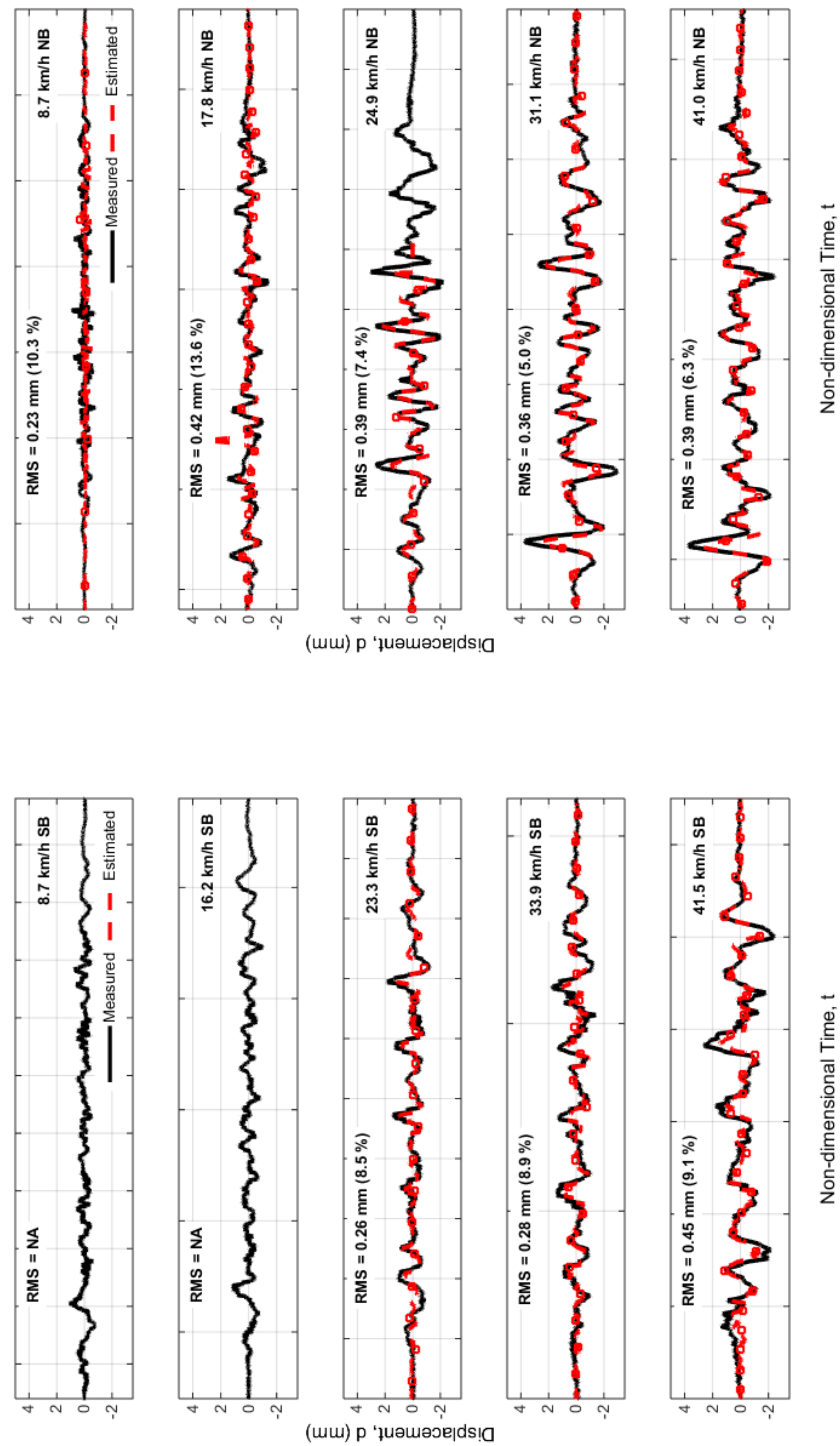


Figure 6.7. Comparison of all WT displacement estimations for pile bent 1 with de-trended displacement measurements: (l) under SB WT; (r) under NB WT

Table 6.1 presents a summary of the maximum estimated and measured displacements for each time history, the maximum displacement range error, and the RMS error and percentage. In general, maximum amplitudes of WSS transverse displacements underestimate LVDT displacements, with the exception of the SB WT traveling at 33.9 km/h (20.8 mi/h), which overestimated displacements by 30.5%. The limits of this estimation were in an order of magnitude of 20%, with the exception of harmonic roll cases and very slow trains. Resonance cases were caused by harmonic roll effects for this type of vehicle and track condition at around 33.9 km/h and 31.1 km/h (20.8 mi/h and 19.2 mi/h) for the SB and NB trains, respectively. The NB WT traveling at 31.1 km/h (19.2 mi/h) underestimated the maximum displacement by 39.8%. Harmonic roll caused errors because of the complexity of non-linear relationships between bridge, track and train at these speeds (Moreu et al. 2014). Another limit to this method is for measuring displacements at low speeds (crawling traffic) such as 8.7 km/h (5.4 mi/h) because they are mostly governed by the pseudo-static component, with 19.8% and 37.5% errors for SB and NB trains, respectively. For all the other six live load tests, errors in displacement estimation are approximately 10% and 20% for SB and NB WTs, respectively. Six out of eight RMS errors were under 10% of their corresponding LVDT peak displacement measurements, indicating the robustness of the WSS measurements.

Table 6.1. Summary of the error of displacement measurements for pile bent 1 under different WT directions and speeds.

Traffic direction	WT speed, v (km/h)	LVDT peak displacement, (mm), A	WSS peak displacement (mm), B	Displacement difference, Δ (mm), (A–B)	Error of estimation, ξ (%) (A–B)/A	RMS error of displacement, RMS (mm)	RMS % (RMS/A)
South Bound (SB)	8.7	1.93	1.54	0.38	19.8	NA	NA
	16.2	2.09	1.97	0.12	5.7	NA	NA
	23.3	3.09	2.82	0.27	8.7	0.26	8.5
	33.9	3.20	4.18	−0.98	−30.5	0.28	8.9
	41.5	4.94	4.48	0.46	9.2	0.45	9.1
North Bound (NB)	8.7	2.28	1.42	0.85	37.5	0.23	10.3
	17.8	3.07	2.40	0.68	21.9	0.42	13.6
	24.9	5.38	4.61	0.77	14.4	0.39	7.4
	31.1	7.31	4.40	2.91	39.8	0.36	5.0
	41.0	6.28	5.03	1.25	19.9	0.39	6.3

For bridge assessment applications, WSS displacements need be amplified by a safety index that takes into account underestimations identified above. WSS estimations under slow trains running at about 9 km/h (5 mi/h) need to be amplified by 40%. WSS estimations under harmonic roll conditions at speeds of approximately 32 km/h (20 mi/h) need to be amplified by 40% to avoid underestimation of displacements. WSS estimations for other speed ranges need be amplified by 20%. Current field measurement methods are limited to eye observation estimations of bridge movements, subjective to the nature of visual observations (bridge inspectors have reported bridge displacements of up to 150 mm (6 in.)); or visual recording of the structure with a laser point from a fixed-remote point with accuracy as low as 3.175 mm (0.125 in.). WSS provide objective metrics to assess bridge condition using displacements. WSS displacements that improve upon current inherent limitations of collecting bridge displacement measurements using visual observation or laser point measurements.

6.3.2 Transverse displacement estimations for pile bents 1, 3 and 4

This section explains the value of collecting approximated values of displacements to assess bridge elements (pile bents) of different condition, and that the quantified errors described above are acceptable for bridge assessment in the field. Pile bent 1 displacements are compared with WSS measurements on pile bents 3 and 4. Pile bents 3 and 4 have WSS that estimate transverse displacements, but they do not have LVDTs. Figure 6.8 shows that displacement estimations for the three pile bents monitored with accelerations during this field experiment (Figure 5.1) capture different pile bent conditions. The estimated transverse displacements for bents 3 and 4 are greater than those for bent 1 (the one from which there are also measured displacements). In particular, pile bent 3 has higher displacements under WTs running in the SB direction. Figure 5.1 shows that pile bent 3 does not have cross bracing. In general, pile bents 3 and 4 are less restrained against transverse movement since they are in the middle of the trestle and further from the fixed condition provided to pile bent 1. Pile bent 1 is the first timber pile bent immediate to the rigid concrete pier supporting the DPG on the North. The estimated displacements also capture evidence of harmonic roll caused by trains running in the neighborhood of 20-32 km/h (15-20 MPH), which is expected for this type of traffic, bridge, and track (Moreu et al. 2014). Railroads could use the displacement estimations of different pile bents within the bridge to prioritize bridge elements maintenance, repair, or replacement. Larger transverse displacement (as high as 100 % larger) are possible indications for additional action/s, including, but not limited to, repairing/adding crossing elements of pile bents with larger displacements within the bridge. Additionally, results from up to 29 field measurements showed that transverse displacements of timber railroad bridges can change up to 300% for the same bridge as the bridge condition changes with time until it is found critical for railroad operations (Moreu et al. 2014). Error displacements of 20% and 40%, once corrected, can indicate different bridge elements condition.

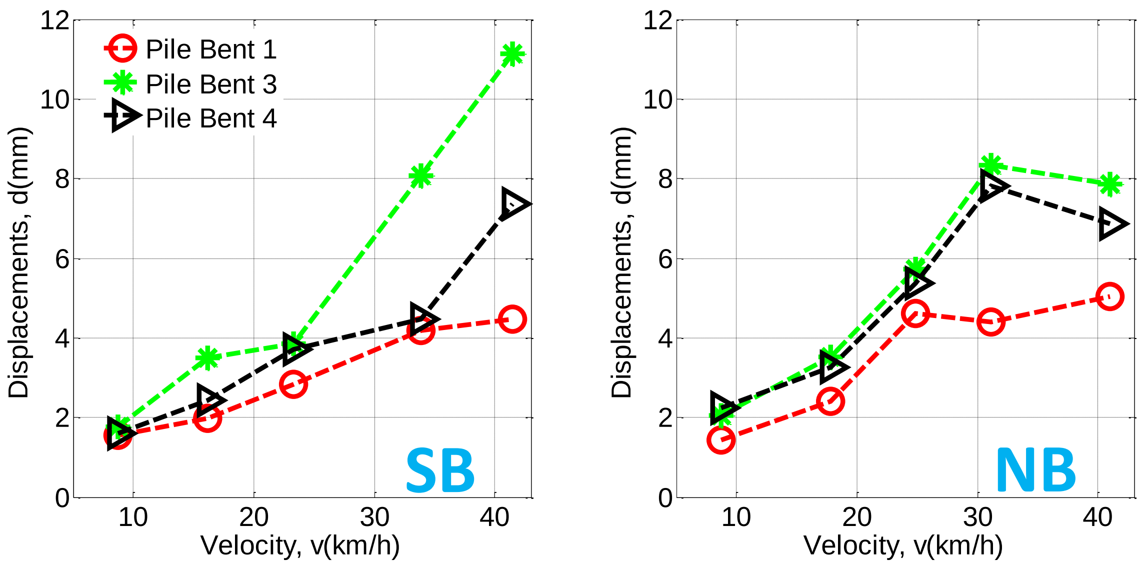
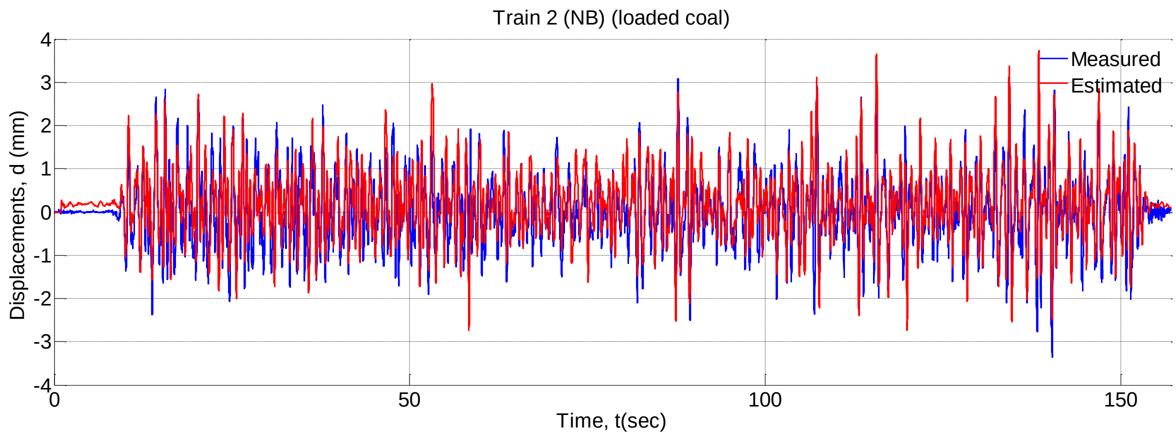


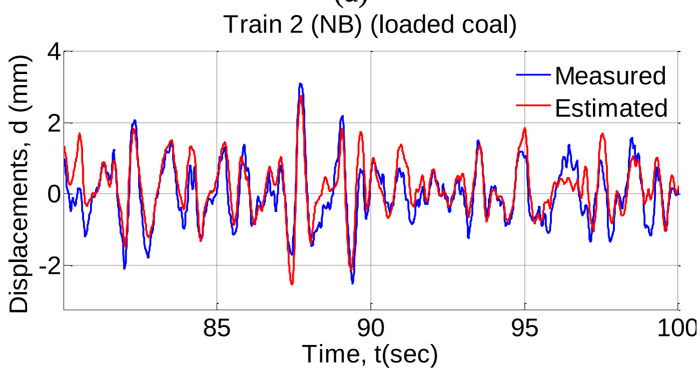
Figure 6.8. Transverse displacement total amplitude estimations vs. WT speeds for three pile bents: (a) under SB WTs; and (b) under NB WTs.

6.3.3 Displacement estimations under revenue service traffic: analysis and results

Results from the estimation of transverse displacements under revenue service traffic also matched well with their respective actual measurements. Transverse displacements and accelerations were measured, and estimated displacements were calculated and compared. Figure 6.9(a) shows the comparison for the entire record, whereas Figure 6.9(b) shows a detailed time history portion of the record for both estimated and measured displacement (which are comparable in amplitude and phase in the time domain). Error of estimation was 17.1% and the RMS error was 0.36 mm (0.014 in.). This error is consistent with six of the live load tests under work trains. Both WSS and LVDT displacements are comparable in amplitude (with a 17.1% error) and phase in the time domain. For bridge assessment applications, WSS displacements need to be amplified by a safety index that takes into account the underestimations identified above. Railroad bridge inspectors can use WSS to estimate displacements under revenue service traffic and include this measurements in their bridge inspection reports.



(a)



(b)

Figure 6.9. Transverse displacement estimation for pile bent 1 under revenue service traffic: (a) full record and (b) detailed time history.

6.4 Conclusions

This chapter shows the application of a new reference-free displacement application for a Class I timber trestle under revenue service traffic. Reference-free displacements can be collected using a less expensive and quicker method than LVDT displacements. Estimates of the transverse dynamic displacements of timber railroad bridges can be obtained from acceleration measurements and were compared to LVDT measurements. Maximum displacement errors of this method under trains running at 8.7 km/h (5.4 mi/h) were 19.8% and 37.5% for SB and NB, respectively. Maximum displacement errors under trains subjected to harmonic roll with speeds at around 33.9 km/h and 31.1 km/h (20.8 mi/h and 19.2 mi/h) were -30.5% and 39.8%, respectively, for the SB and NB trains. The rest of the six live load testing errors in reference-free displacement were generally below 10% and 20% for SB and NB WTs, respectively, and the RMS errors of all measurements were under 0.45 mm (1/64 in.). With the exception of trains at slow speed or under harmonic roll, reference-free transverse displacements at a critical bridge location (as identified by the railroad) were consistently estimated using accelerations collected by WSS. WSS can readily help identify bents with deficient bracing. Further accuracy of the estimation using WSS can be attain incorporating multimetric sensing that can capture pseudo-static responses of bridges under trains, adapting work from other researchers for measuring dynamic vertical dynamic loads of highway bridges. The size, low cost, portability, low power consumption, and ease of installation of WSS, in conjunction with the results of this research, may allow for more frequent use of displacement measurements in helping assessing the health of timber railroad bridges and their elements (pile bents). Collecting displacements from similar bridges of different condition can provide metrics describing their performance under revenue service traffic. Using evidence of transverse displacements, especially those indicating changes in bridge condition, railroads can determine and include limit(s) on transverse displacements in their assessment practice and/or the AREMA manual, in addition to the current AREMA limit on normalized vertical displacements under trains. Future research will include measuring a larger number of trains and bridges to build robustness and provide more evidence of the proposed reference-free displacement methodology for bridge assessment to inform transverse displacement limits.

CHAPTER 7 EXAMPLE OF CONSEQUENCE-BASED MANAGEMENT OF RAILROAD BRIDGE NETWORKS

This chapter describes the components of the framework for the example of a network of timber railroad bridges. In this example, MRR decisions of timber railroad bridges are prioritized using displacements, because timber railroad bridges are 24% of the current inventory by foot (FRA 2008). The Canadian National Railway (CN) owns the longest timber railroad bridge in America, the Illinois Central (IC) Bonet Carrè Spillway Bridge, with 11,735 feet, by New Orleans, Louisiana (Wanek-Libman, 2014). This research develops this framework to prioritize MRR decisions of timber railroad bridges in North America, because forecasted railroad operations predict increasing car loads from 286 kip/car to 315 kip/car, and none of the timber railroad bridges are designed for any load augmentation. The estimated cost of replacing the timber population of railroad bridges ranges between $15B to $25B and the funds are not there. Fragility curves are determined with SL, and updated using field data. Subsequently, an example determining operational costs and optimal MRR decision is provided.

7.1 Fragility curves for bridge condition assessment

This research develops industry informed fragility curves of SL based on measured bridge performance. Because there is limited data available, the SL of the fragility of railroad bridges are defined by railroad bridge experts, who are familiar with the serviceability and safety limits. These experts represent multiple areas of expertise within the railroad bridge engineering community. To determine the limits of displacements associated to different service limits a survey of experts was conducted following the Delphi technique. Twenty experts in railroad bridge structural engineering were asked about the limits of serviceability of railroad bridges using displacements.

There were members from the private sector, railroad industry, government agencies, and universities.

To determine the SL of railroad bridges under revenue service traffic, following the Delphi method, a second survey was conducted with a reduced group of experts showing limits of displacements with service thresholds for a specific bridge class and geometry and LOS. The following example shows the fragility curve for a typical timber railroad bridge of class 2 under freight loaded trains running at approximately 25 mph. The thresholds are normalized relative to the height of the timber pile, assuming a normal height range between 10 and 50 feet, and similar, standard bridge conditions (i.e. tangent track, no grade, ballasted track, symmetry, etc.) Figure 7.1 shows the cumulative probabilities for a given displacement requiring a slow order estimated by these experts' opinions. Bridge experts of one railroad validated the representativeness of these hypothetical cumulative probabilities based on internal (and reserved) company operations and MRR policies. Figure 7.2 shows the probability density distribution and cumulative probability, respectively, of a slow order. The data is approximated with the best fits for lognormal distribution.

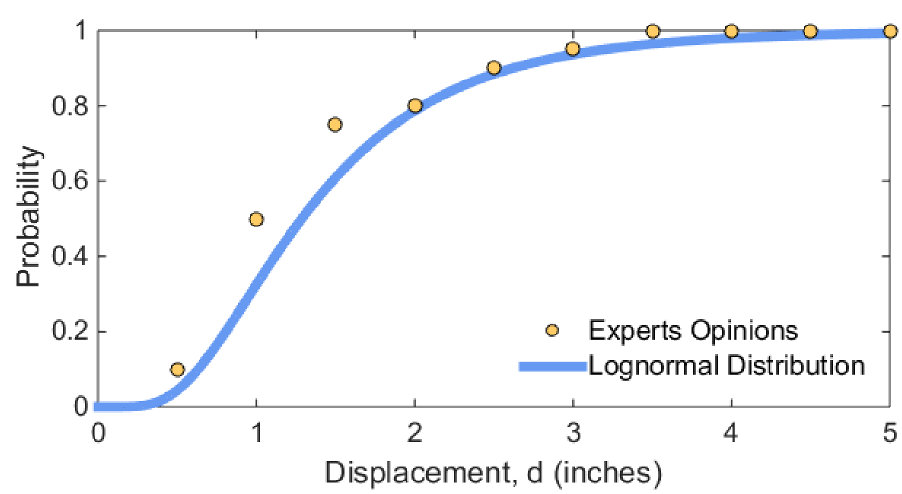


Figure 7.1. Cumulative probability of railroad experts' estimated slow orders and lognormal fit.

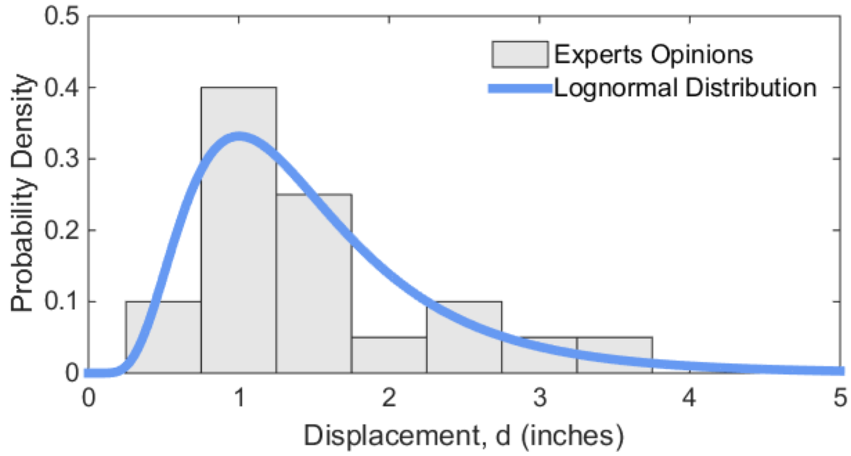


Figure 7.2. Probability distribution of experts' opinions of slow order and lognormal fit.

Figure 7.3, Figure 7.4, Figure 7.5, Figure 7.6, and Figure 7.7 show the fragility curves for timber railroad bridges of Class I, Class II, Class III, Class IV, and Class V, respectively. *H* is the height of the pile bent.

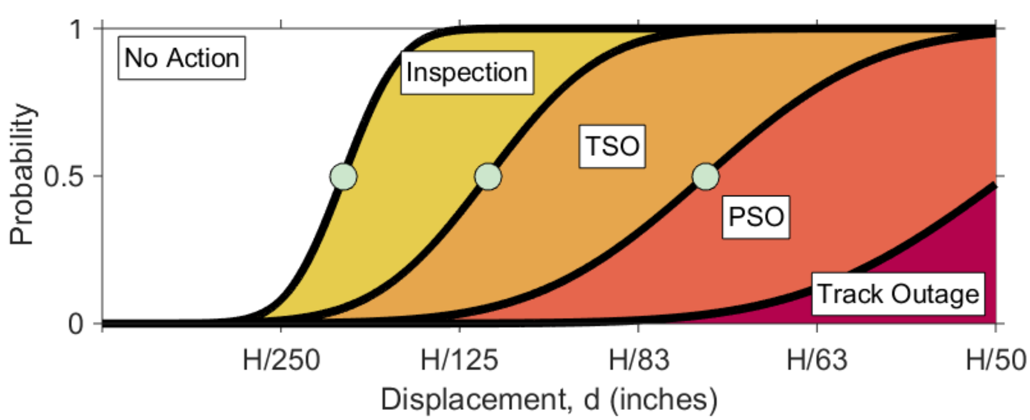


Figure 7.3. Fragility curves for Class I timber railroad bridges.

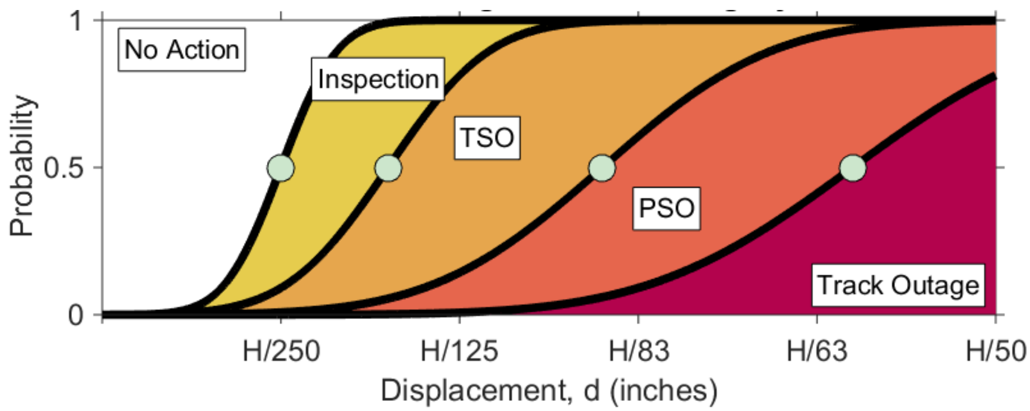


Figure 7.4. Fragility curves for Class II timber railroad bridges.

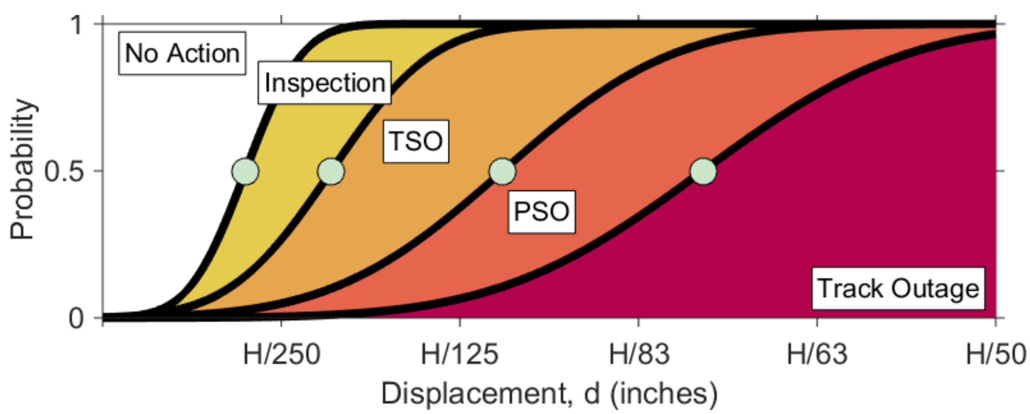


Figure 7.5. Fragility curves for Class III timber railroad bridges.

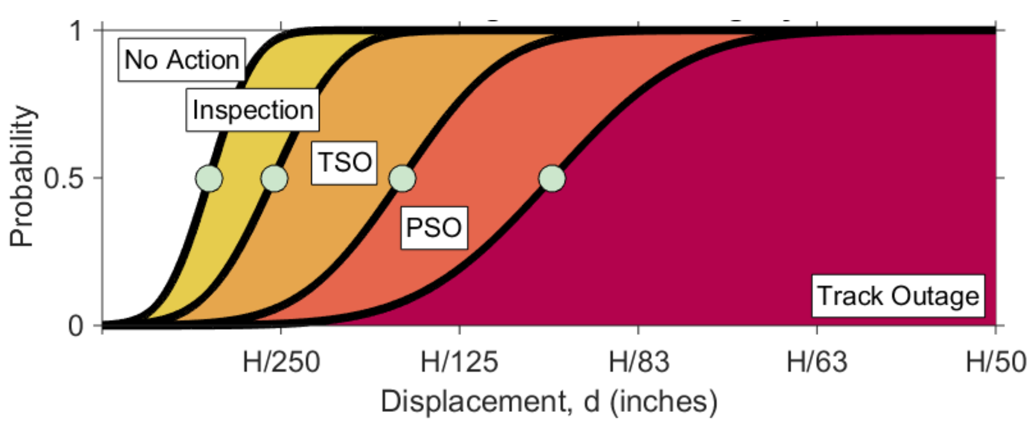


Figure 7.6. Fragility curves for Class IV timber railroad bridges.

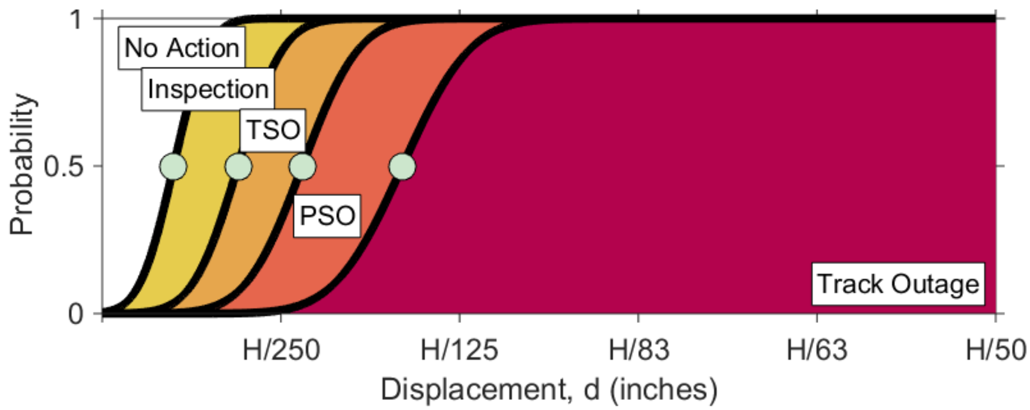


Figure 7.7. Fragility curves for Class V timber railroad bridges.

7.2 Campaign monitoring and bayesian updating

The campaign monitoring provides the maximum displacement measured under revenue service traffic to obtain the probability of service levels. For this example, it is assumed that all of the pile bents of a given timber railroad bridge have been measured under the train crossing, and that the maximum displacement represent the health of the timber railroad bridge. The displacement can be used to update the probability of service limit-states based on measured data. Bayesian theory can obtain the updated probability of the different service limit states using the maximum displacement under a given train and the assumed state of the bridge. The probable bridge state can be updated using the displacement data collected during the campaign monitoring inspection on the bridge. Using Equation (5.4), the updated probability of a Service Limit-state for a given displacement is calculated. The first term is a normalization factor that converts the final estimation into probability. The second term integrates the different areas of intersection between the believed displacement distribution of the bridge and the prior fragilities (see Figure 4.9. Probability distribution based on the believed current state of the bridge for updated bridge state assessment using measured data.). The third term calculates the probable service limit states based on the assumed displacement distribution and the maximum measured displacement. Using the measured maximum displacement collected from a bridge under construction (Moreu et al. 2014), the maximum displacement was 1.8 inches. Using an assumed distribution of Service Limit-states for this bridge, the different probabilities are calculated (Figure 7.8). The final updated probabilities for the different Service Limit-states are shown in Figure 7.9.

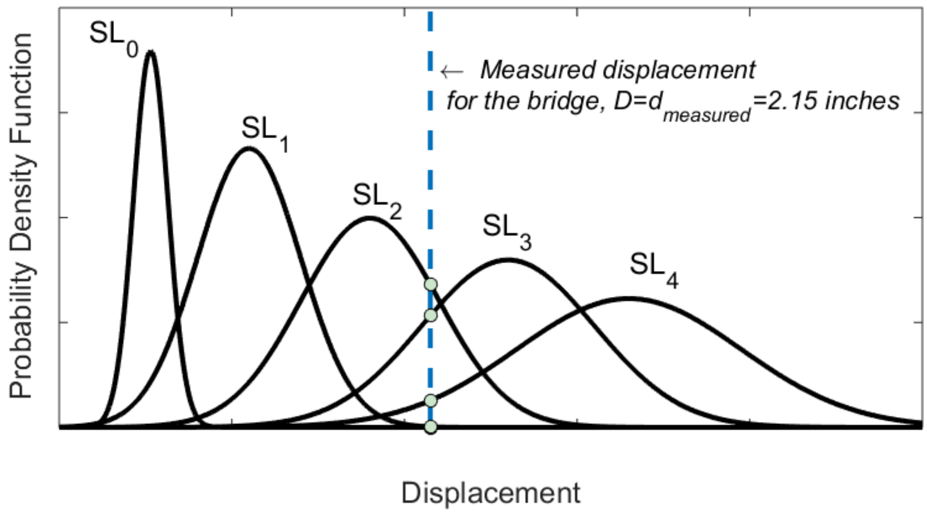


Figure 7.8. Distribution of displacements $P^{\text{prior}}\left(D=d_{\text{measured}} \middle| SL=k, Z=j\right)$, given the service limit state SL_k and the track class Z_j.

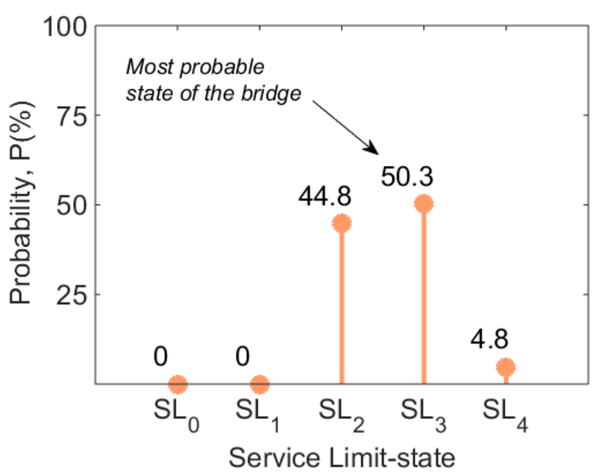


Figure 7.9. Updated distribution $P^{\text{updated}}\left(SL=k \middle| D=d_{\text{measured}}, Z=j\right)$ of SL_k, given the measured displacement $D=d_{\text{measured}}$ and the track class Z_j.

7.3 Operational costs

The following example estimates the expenses of one timber railroad bridge of Class IV (200 feet long and 20 ft tall) in the main line with a measured displacement of 2" under regular traffic (10 trains/day). The cost of timber railroad bridge replacement as determined by the railroad is $6,500/feet for normal construction conditions and access, and the cost of a slow order per train per hour is $261 (Lai and Barkan, 2009). These costs do not include traffic interruptions, and only costs of fuel and crew. This figure is assumed as the cost of permanent slow order because is under planned circumstances and included in the time table. The cost for unplanned delays is estimated using variable train delay costs for different route lengths and assumptions by Lovett et al. (2015). Slow order and the track outage operational costs per train and hour are $1,438, averaging manifests and intermodal estimated costs for a general case. Averaged TSO and PSO delay times/train are 10 and 2 minutes, respectively, based on railroad operations from a Class I railroad. Table 7.1 shows the computation of each component for each SL_k. Probabilities for $P(SL_0)$, $P(SL_1)$, $P(SL_2)$, $P(SL_3)$, and $P(SL_4)$ with (d=2 in.), are:0.0001, 0.03, 23.82, 56.84, and 19.31%, respectively. The total cost for bridge replacement is $6,500/ft*200ft=$1.3M. The estimated OE and LR are shown for each SL_k. In this example, OE were low and LR were high. The total annual estimated OC is $844,317, which is based on excessive transverse displacements for a bridge of class IV in the main line and the proposed consequence-based assessment for 2 inches.

Table 7.1. Estimated Operational Costs breakdown example for one bridge.

	$P(SL_k)(\%)$	$OE(\$)$	$LR(\$)$	$OC(\$)=OE+LR$	$<OC_k>(\$)=P\left(SL_k\right)\cdot OC_k$
***SL*₀**	0.0001	0	0	0	0
***SL*₁**	0.03	25,000	0	25,000	7.50
***SL*₂**	23.82	26,000	575,200	601,200	143,205
***SL*₃**	56.84	65,000	762,120	827,120	470,135
***SL*₄**	19.31	130,000	1,106,080	1,196,080	230,918
	$<OC^n>=\sum_{k=1}^{K} P\left(SL=k \middle| D=d_{measured}, Z=j\right)\cdot OC_k^n$				844,317

Using the fragility curve for a Class IV track and Equation 6, operational costs are calculated for different displacements for the same bridge (Figure 7.10). This figure explains why some railroad companies can control smaller displacements with minimum operational costs, but there is a performance threshold from where the costs increase should be corrected (MRR costs increase when they are not planned). In this figure, under 0.5 inches, operational costs could be acceptable by the railroad (under 45,000$/year). However, from 0.75 inches and above operational costs are over six figures. Consequently, in this example, 0.5 inches could be the threshold for consequence-based performance.

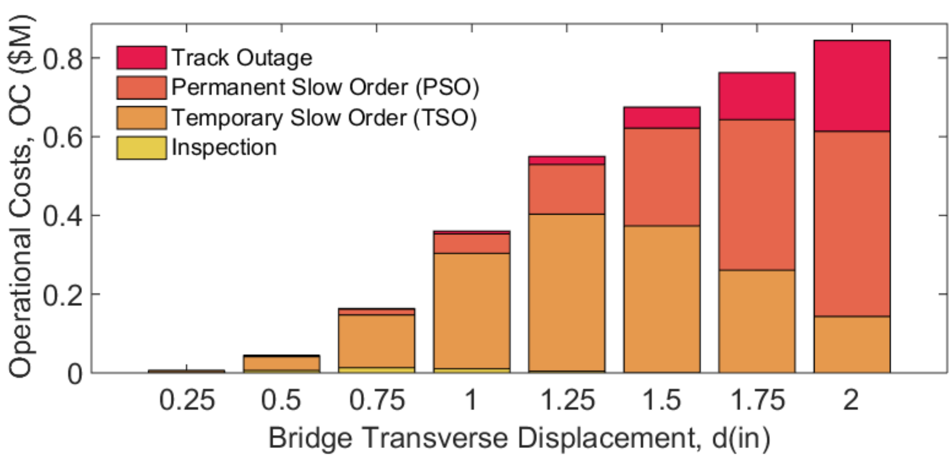


Figure 7.10. Operational costs for the same bridge (class 4) under different levels of displacement.

7.4 Consequence-based management to inform MRR decision for multiple bridges

To illustrate the potential of this tool, an example of MRR decisions is presented in one example. For one given year, an example of 50 bridges of a railroad company illustrates this example. To better understand how MRR decisions are made, Table 7.2 presents 10 different bridges belonging to different locations within the network. The information provided in the example is taken from recent timber railroad bridge replacements in the Midwest, but their specific properties are modified to maintain the confidentiality of the company providing this information. All bridges are in the main route (high traffic), but different traffic levels are shown for different bridges. Bridges a, c, e, h, i, and j have a less track class than the subdivision they belong. The inspection reports recommend different MRR decisions that are included in the table. MRR decisions are based on structural capacity. MRR costs are calculated based on the length of the bridge and Table 7.2. MRR costs for each bridge are independent of the service condition of the bridge and are shown in Figure 7.11.

Table 7.2. Bridge network information for MRR decisions prioritization.

Bridg e Label	Subdivision Name	Subdivision Track Class, SC (#)	Bridge Track Class, BC (#)	Length, L (feet)	Traffic, T (trains/day)	MRR
a	South Bend	5	4	200	25	Replacement
b	Freeport	4	4	400	20	Maintenance
c	Edgewood	4	2	300	15	Replacement
d	Bluford	3	3	300	10	Repair
e	Stonefort	3	2	400	5	Repair
f	Lowes	3	3	200	10	Repair
g	Mile Long	3	3	4200	20	Maintenance
h	Loosahatchie	4	3	1000	20	Repair
i	Fulton	4	3	300	15	Replacement
j	Yazoo	4	2	600	15	Replacement

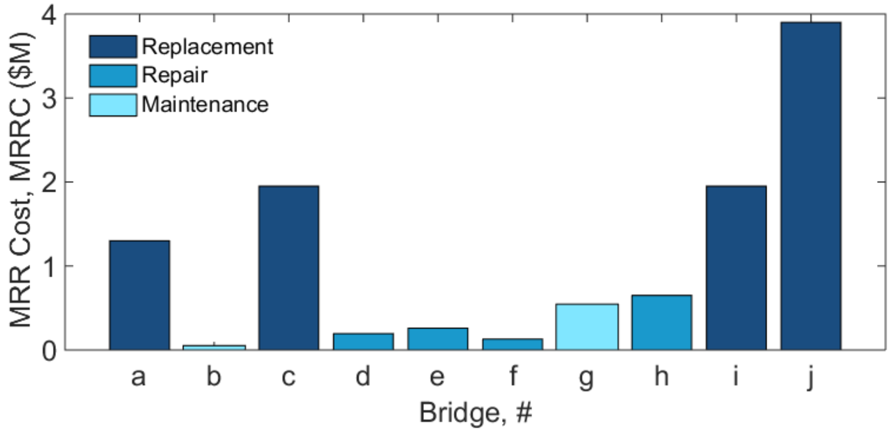


Figure 7.11. MRR decisions cost for a work program based on structural capacity.

7.4.1 MRR decisions without information about displacements

If there would be no budget limits, the railroad would replace the entire population of timber bridges to increase the capital, the capacity, and the safety of their network: replacing timber railroad bridges eliminate risks and increase capacities. However, replacing the 10 bridges costs $51.35M which is neither affordable nor urgently needed. The cost of the recommended MRR decision from Table 7.2 (ten bridges) is $10,93M. For a $10M limit for the 10 bridges, one policy for MRR decision without consequence-based framework is shown in Figure 7.12. The first MRR decision is to prioritize replacement of bridges causing PSO to the main line, which are a, c, i, and j (South Bend, Edgewood, Fulton and Yazoo). Bridge replacements increase capital to the company. Secondly, bridge h (Loosahatchie) is repaired. All other five bridges are neither repaired nor maintained because the MRR budget has to stay under $10M. Without using the consequence-based framework, this MRR decision upgrades five bridges.

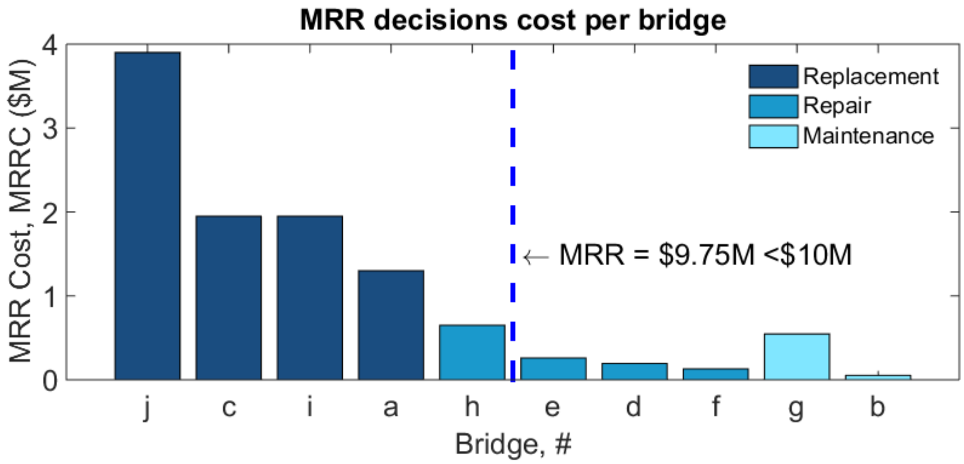


Figure 7.12. Best use of MRR budget without displacements.

7.4.2 MRR decisions and operational costs using displacements

The proposed framework can be used to minimize expected costs for a given network. Table 7.3 shows a hypothesis of displacements for the given bridges, based on traditionally observed levels of displacements reported by a Class railroad I for bridges in the main line within the work program and past timber railroad bridge live load testing (Moreu et al. 2014; Moreu et al. 2015). The bridge height are generated to represent a realistic population of bridges. The displacement index, i (d/H) goes from as low as 1/1000 (very small) to 1/48 (very large). Based on these displacements and the fragility curves, operational costs are calculated. Operational costs are assumed to be annual. Figure 7.13 shows the operational costs per bridge for the MRR policy without displacement information. The total operational costs for this policy is OC (e) + OC (d) + OC (f) + OC (g) + OC (b) = $2.11M.

Table 7.3. Bridge displacement hypothesis.

Bridge Label	Subdivision Name	Bridge Height*, h (ft)	Displacement, d (in)	Index, i d/H*12
a	South Bend	16	2.25	1/111
b	Freeport	62	0.25	1/1000
c	Edgewood	14	2.75	1/91
d	Bluford	36	1	1/250
e	Stonefort	15	2.5	1/100
f	Lowes	13	3.25	1/48
g	Mile Long	42	1	1/250
h	Loosahatchie	50	1	1/250
i	Fulton	21	2	1/125
j	Yazoo	9	2.25	1/111

* Maximum pile bent height at the point of measurement

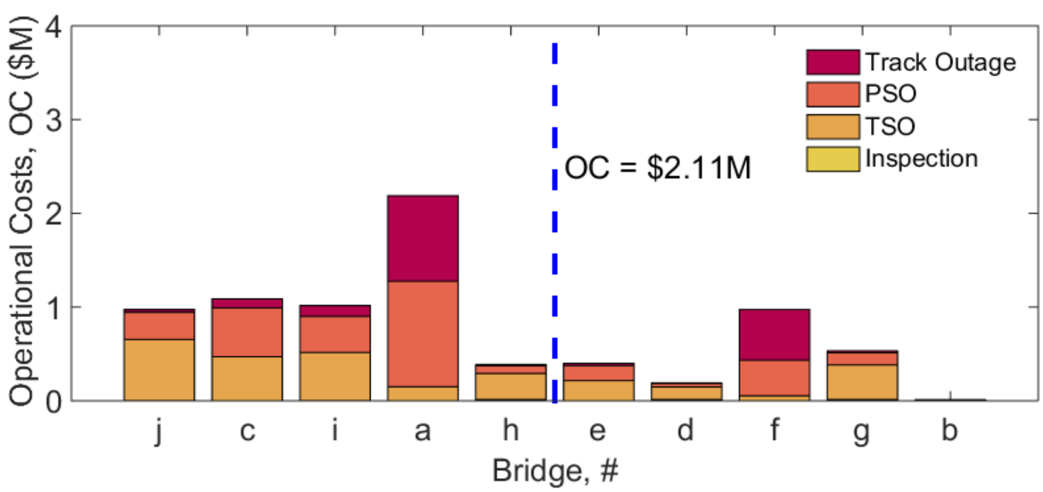


Figure 7.13. Operational costs for MRR policy without displacement information.

The operational costs of decisions using traditional information are compared to the operational costs of decisions using fragility curves. Using the operational costs information for each bridge, the sequence of MRR can be reorganize to do MRR activities in those bridges with higher operational costs. For example, operational costs of bridge (f) are significant (>$0.97M/year), where MRR for this bridge are small ($0.13M). For a given limited budget, bridge MRR decisions can be selected minimizing estimated costs of decision policies at the network level. Figure 7.14 shows the sorting of operational costs to inform MRR decisions, and

Figure 7.15 shows the MRR policies associated to informed decisions using fragility curves. The result is that for the same budget of $10M, the new policy has an operational costs of OC (e) + OC (h) + OC (d) + OC (b) = $0.99M (53% saving from $2.11M, see Figure 7.13).

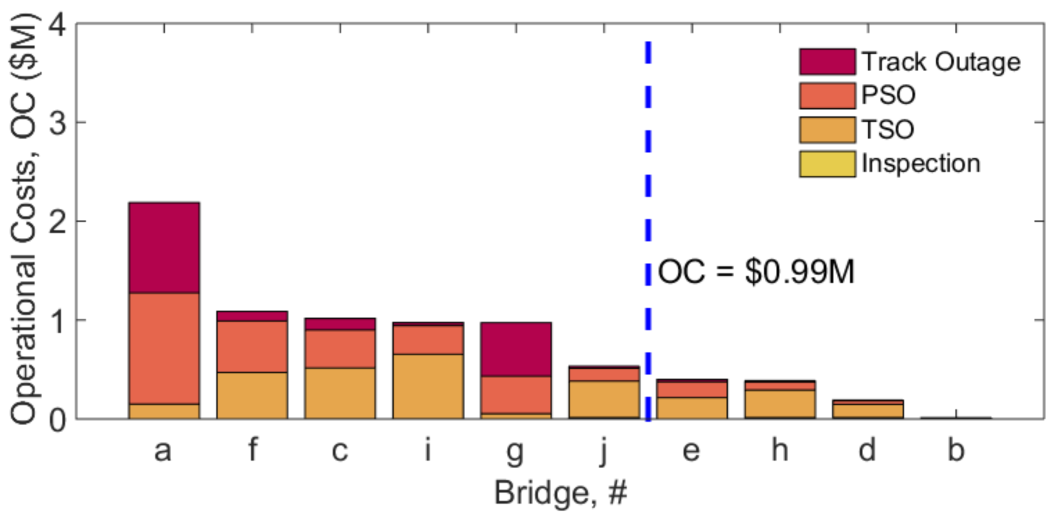


Figure 7.14. Operational costs sorted in descending order.

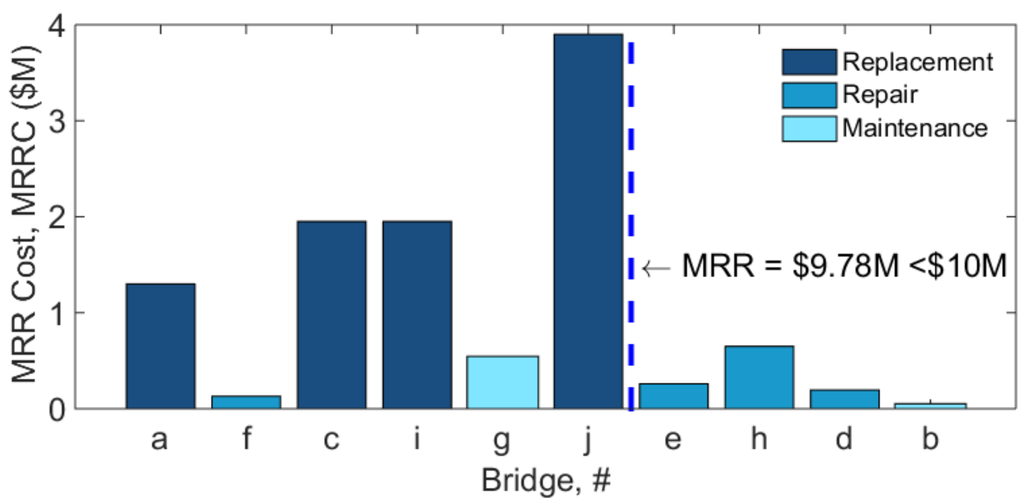
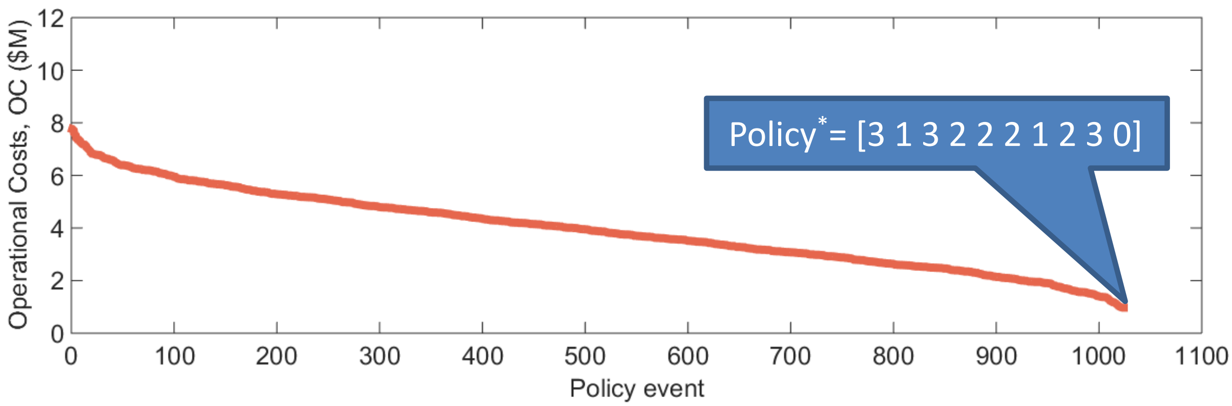


Figure 7.15. MRR policy based on minimization of operational costs per bridge using displacements.

7.4.3 Combination of MRR decisions costs and operational costs using displacements for a given year

The proposed framework can be used to minimize expected costs for a given MRR policy, but the MRR budget decisions can be improved if the MRR are modified minimizing operational costs. Having the operational costs associated to each MRR, a single change of MRR policies can be made to delay the MRR for the future (capital saved by the company if it can be applied in the future without significant cost to the operations). If operational costs are small, delaying MRR decisions for one bridge for one year can save significant capital investment to the railroad. This option is calculated for the 10 bridges. The possible policies are 210=1,024. The total operational costs for all MRR are sorted and the minimum operational costs is calculated to be $ 0.97M (Figure 7.16). This value corresponds to operational costs (bridge j) because this policy is doing MRR in all 9 bridges, and delaying MRR in bridge j for the future. If bridge j would not be replaced this year, the railroad would save $3.9M in MRR that could be used for other operations within the company, at the operational costs of $0.97M. This decision implies 39% saving in MRR budget. Additionally, the railroad would only have $0.97M of operational costs (which is 54% less than the original operational costs of $2.11M). Results show that the proposed framework can be optimal making consequence-based MRR decisions. Figure 7.17 shows all possible MRR costs and operational costs for the 1024 options under $10M for year 1. MRR can be minimized for a given fixed operational costs for each year, and the railroad can plan decisions of the most efficient use of $ MRR/year. The lighter color of the circles show lower $ operational costs/year. Optimal MRR policies are chosen with lower $ operational costs/year. For example, MRR Costs of less than $2M implies operational costs over $5M, whereas the MRR Costs larger than $2.2M yields less than \$3.5M operational costs (\$1.5M less). For this example, the total network cost of an optimized MRR decision can save almost \$1.5M/year.



* Policy for 10 bridges (from a to j): [3]: Replacement, [2]: Repair; [1]: Maintenance; [0]: No action.

Figure 7.16. Lowest operational costs including all possible modified MRR policies.

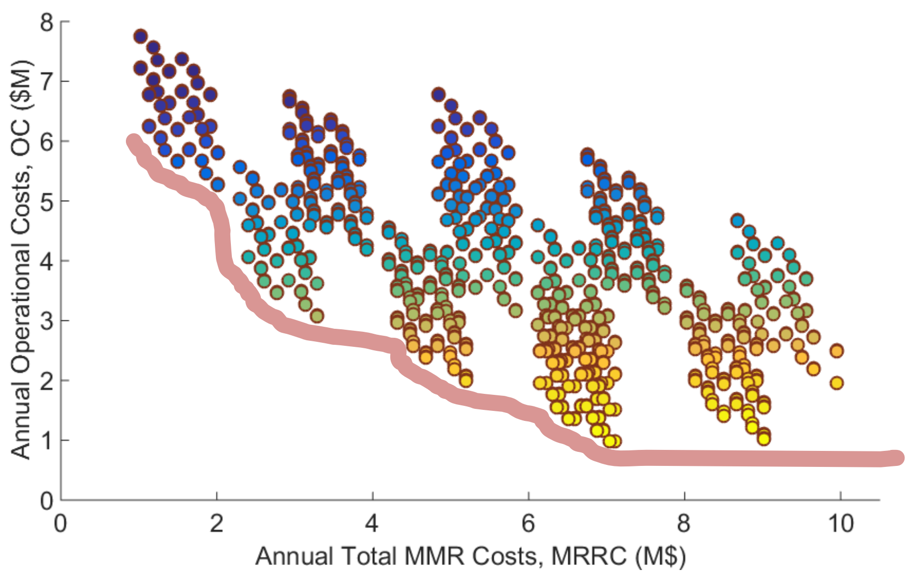


Figure 7.17. Operational Costs vs. MRR Cost for all possible MRR policies.

7.4.4 Generalization of MRR decisions for multiple years for 50 bridges

The benefits of measuring displacements can be also applied to multiple year scenarios (short term). MRR policies can be modified for a fixed annual MRR cost and minimizing operational costs. For three consecutive years, the different MRR and operational costs values are calculated and added together for a network of 50 bridges of similar properties as those described in Table 7.2. Displacements associated to these 50 bridges are similar to those Table 7.3. Bridges that are replaced in year 1 will not have any operational costs on the future. Bridges that are not replaced in year 1 have a minimum MRR cost associated to maintenance. An interest rate of 6% is applied following recommendations from Frangopol et al. (2001). The result of minimizing both MRR budgets and operational costs annually yields to a total of \$77.3M in three years (see Figure 7.18). The optimum MRR policies identify the minimum investment of \$6.5M/year. Local maximum total network expenses were at M_{budget}=\$0M and M_{budget}=\$51.5M, being \$108.7M and \$133.3M, respectively. Savings were between \$31.4M and \$56M (29% and 42%, respectively). The entire network has 50 bridges in the work program. Savings can be larger in networks with higher number of bridges and for longer time-frame decision scenarios. This method permits railroads to make MRR decisions incorporating operational costs caused by traffic at the network level. These fragility curves can also be developed for other bridge types such as pin-connected trusses where displacements are also identified to be of interest to inform railroads about the bridge service condition under revenue service traffic.

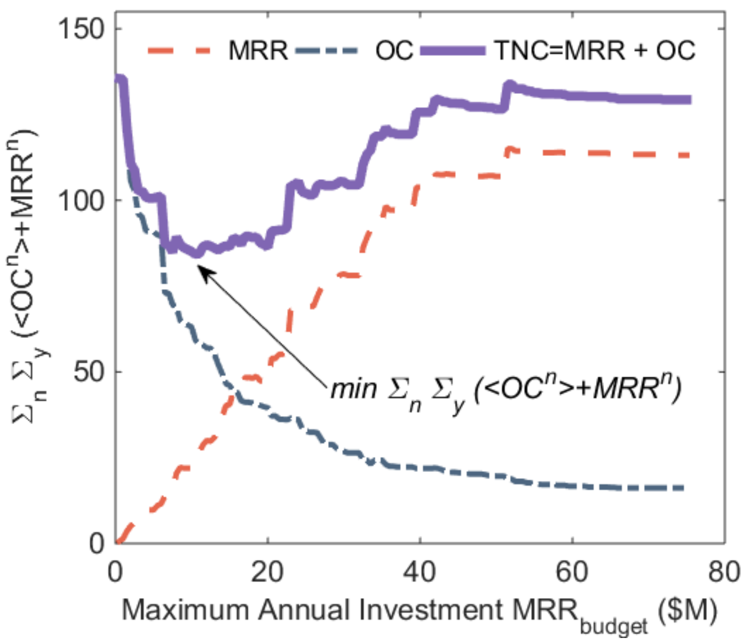


Figure 7.18. Minimization of total MRR costs and operational costs based on a fixed annual operational costs for three years.

7.5 Conclusions

This chapter develops a consequence-based framework that prioritizes MRR decisions of railroad bridge networks, estimating the operational costs of bridge SL given bridge responses. The goal is to minimize the expected value of the total network cost. Critical to the framework is the ability to assess bridge service condition. Railroads can collect objective performance information of the bridge service condition under revenue service traffic using wireless sensors. Performance of railroad bridges can then be used to prioritize the infrastructure. Fragility curves relate the measured bridge performance with a SL and also calculate the operational costs associated for each specific bridge and location. The railroad can prioritize the upgrading of their railroad bridge networks finding the optimal MRRs that minimized operational costs. This paper provides one example focusing in timber bridges because 24% (by length) of bridges are still timber in the U.S. The initial source of information to determine the SL uses experts' opinions. Railroads can use this framework to prioritize decisions on MRR at the network level by minimizing total network costs. For this example, savings on total network costs for a bridge network of 50 bridges are between $31.4M and $56M (29% and 42%, respectively). Wireless Smart Sensors (WSS) measurements can inform of bridge condition with moderate effort and provide evidence to inform prioritizing or delaying MRR decision. As part of future work, using data collected in the field of timber railroad bridges of known condition, the fragility curves can be updated using a Bayesian approach. Additional future work includes measuring both track and bridge responses under different service limit-states. Finally, using evidence of transverse displacements of changes of bridge serviceability can assist to determine and include limit(s) on transverse displacements in their assessment practice and/or the AREMA manual, in addition to the current AREMA limit on normalized vertical displacements under trains. This framework provides an intelligent use of bridge response information to inform consequence-based management of railroad bridge networks, minimizing railroad bridge total network costs.

CHAPTER 8 VALIDATION OF SHM TECHNIQUES USING WSS FOR RAILROAD BRIDGES

The primary objective of this chapter is to apply portable, cost-effective, and practical SHM system using WSS to prove their potential in the context of railroad bridges assessment. The system adapts wireless sensor technology developed at the University of Illinois as part of the ISHMP (Figure 8.1). This research will demonstrate that railroad bridge responses can be collected efficiently and quickly in the field using wireless smart sensors, and analyze this data to predict quickly structural responses of the bridge at different locations within the bridge. To show the direct applicability of this concept within the railroad environment, Illinois partnered with CN to carry out the technical scope of this research. Ultimately, this research is expected to provide railroads with new objective information about the in-service performance of their bridges that can enhance inspection quality, improve safety, reduce maintenance costs, and help to prioritize bridge repairs and replacements.

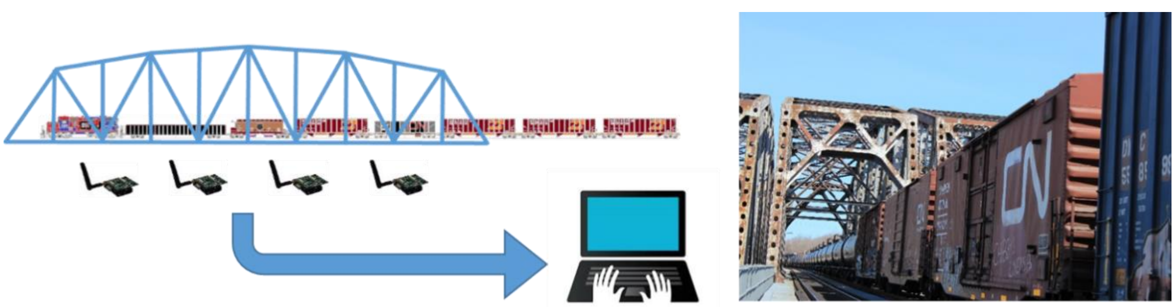


Figure 8.1 Concept of proposed wireless sensing system.

8.1 Motivation

Numerous analytical studies have been conducted considering the response of bridges under revenue service traffic. The goal was to gain a deeper understanding of the critical loads and speeds for specific bridges. However, few efforts have been conducted to experimentally validate these models, limiting their predictive power and ability to improve bridge design and prioritize bridge repair and replacement with informed decisions. The scarcity of experimental results has been due, in part, to the high cost of instrumenting a bridge.

WSS offer an opportunity to provide a portable tool that can quickly be installed, used, and easily removed for use on other bridges by railroad personnel. Additionally, because of the onboard computing capabilities of the sensor nodes, real-time, practical information can be provided that can be interpreted right at the bridge. Developing a campaign monitoring system specific for railroad bridges environment using WSS addresses, but is not limited to, the following current needs:

- Safety – use of regular campaign monitoring of bridges to ensure railroad safe operations.
- Economical / Managerial - bridge replacement prioritization requires quantifiable data about the bridge population to enable rationale decision making and budget allocation.
- Planning and Transportation - railroad transportation and capacities can be maximized by better identifying the current structural capacity of the bridge population within the network.
- Institutional - regulatory recommendations, incentives, and penalties associated with bridge monitoring (and liability consequences) to improve the safety of railroad operations.

8.2 Objective

This research shows that WSS can be effective and inexpensive tools to monitor traffic loads and bridges responses under revenue service traffic. A CN double-track steel truss bridge over the Calumet River on the South side of Chicago, Illinois was selected to validate the practical implementation of WSS under revenue service traffic. Results of this research include evidence that magnetic strain gages can effectively and accurately collect strain data from structural elements. This experiment found that train speeds do not affect Impact Factor (IF) measurements for this railroad bridge. Finally, this data was used to calibrate an FE model that used input loads for bridge response estimations. The final result is an autonomous monitoring strategy for railroad bridges using WSS. Because Chicago is the busiest rail hub in the United States (CREATE, 2014), this experiment proves that WSS are effective tools to safely collect train loading characteristics and bridge responses under revenue service traffic.

8.3 Bridge description

This dissertation validated the viability of WSS for railroad bridges using a real bridge under revenue service traffic to validate the applicability of the proposed system. The selected bridge (Figure 8.2) is a double-track steel truss located on the South side of Chicago, Illinois at mile post (MP) 16.9 over the Little Calumet River, a 94.6 meters (310 feet - 4 inches) span with both Amtrak and freight traffic on both directions: North Bound (NB) and South Bound (SB).

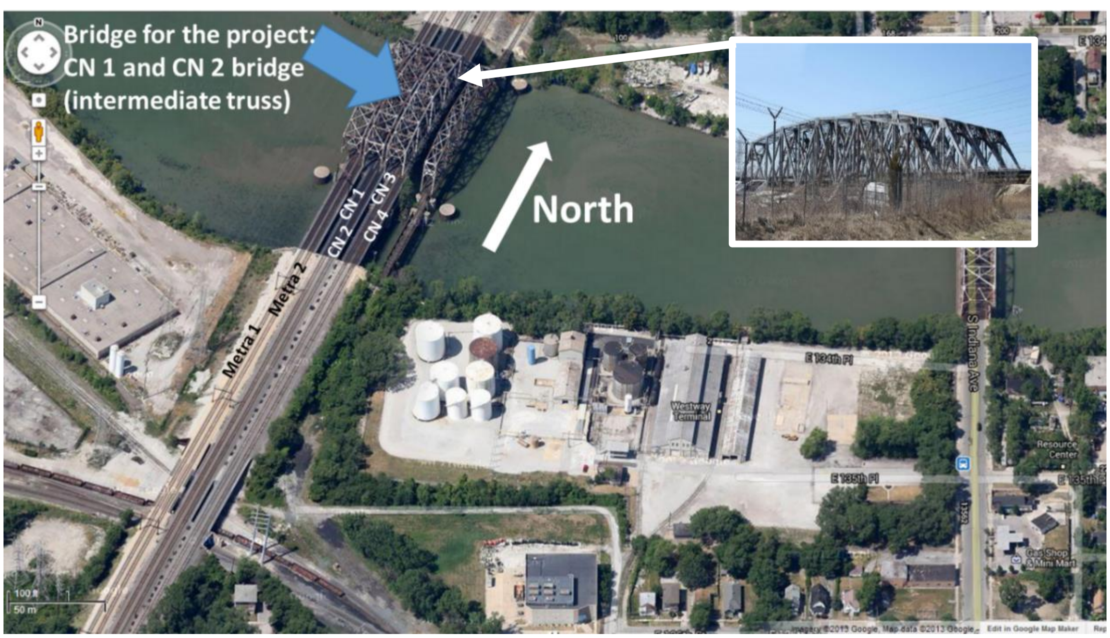
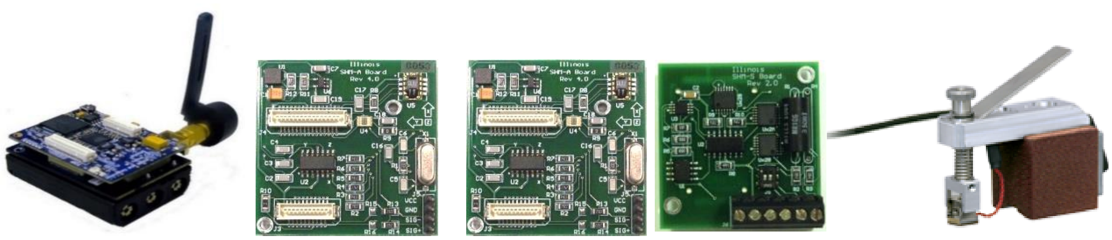


Figure 8.2 Bridge over the Little Calumet River (near Chicago, IL)
(Google Maps, 2012).

The bridge was designed in 1960, following the 1956 American Railway Engineering Association (AREA) recommended practices and specifications for steel railroad bridges. CN built the bridge in 1971 with an expected service life of 100 years, with live load E-72 Cooper and unit stresses for axial tension and compression of 124.1 MPa (18,000 psi) and 103.4 - 0.0017 (L/r) MPa (15,000 - 0.25 (L/r) psi), where L/r is the slenderness of the member under consideration. The most recent inspection reports of this bridge proved that the current state of the structure was not changed from the date of construction. This bridge has a large amount of traffic per day (approximately twenty trains, including both freight and Amtrak traffic). This application of WSS supports the advantages of using WSS technology within heavily transited railroad environments.

8.4 Instrumentation

The WSS instrumentation for this experiment can collect instantaneous bridge responses to loads close to real-time. This research was conducting using the Imote2, a WSS platform developed by Intel (Figure 8.3(a)) (ISHMP, 2014). The Imote2 includes a high-performance X-scale processor (PXA27x), 256K static random-access memory, 32MB Flash memory, and 32MB synchronous dynamic random access memory, which enables the intense onboard calculation required for SHM applications, as well as storage of longer measurements. Sensor boards are stacked on the Imote2 via two connectors to facilitate sensing with the Imote2. Sensor boards include a general-purpose accelerometer board and a strain sensor board for the Imote2, called SHM-Acceleration (SHM-A) (Rice et al. 2009) and SHM-Strain (SHM-S) (Jo et al. 2012) sensor boards, respectively (Figure 8.3(b), Figure 8.3(c)). To improve the campaign monitoring of railroad bridges by enabling easier and simpler measurement of strain, the Illinois research team explored the use of a magnetic strain gage for both rail and structural strain (Figure 8.3(d)). The strain was measured using the magnetic strain gage (model FGMH-2A) from Tokyo Sokki Kenkyujo Co., Ltd. (2014). There were two wireless smart sensors networks for this project: (i) accelerations, and (ii) strains. Accelerations and strains were sampled at different rates, 50 and 100 Hz and 280 Hz, respectively, to maximize their different purposes. Accelerations were used to calibrate a Finite Element (FE) model built to estimate strains of other members under train loads. Strains were used to estimate input loads and to measure bridge structural elements response to train loads. Figure 8.4 shows the sensor layout including the base station PC location and the structural diagonal element (L4-U5) being monitored. Table 8.1 shows the technology assessment justification, information to be collected, sensors, and WSS locations. This WSS system meets the information needs for campaign monitoring.



(a) (b) (c) (d)

Figure 8.3 (a) Imote2 with an external antenna and sacked on a battery board, (b) SHM-A sensor board, (c) SHM-S sensor board, and (d) magnetic strain gage.

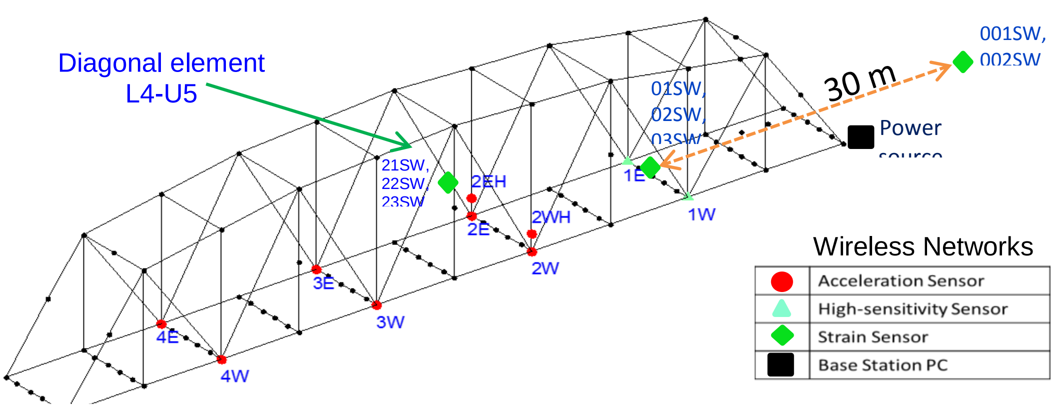


Figure 8.4. WSSs layout.

Table 8.1. Monitoring objectives and WSS strategies.

Objective	Information	Wireless Sensor type	Bridge location	Notes
Loading properties	Rail strain	Strain	Both inside and outside the bridge	Measures loading data from railroad cars, tests suitability of magnetic strain gage for rail applications
Dynamic properties	Accelerations	Accelerometers	Main nodes (both planes)	Installed at two different planes to capture 3D modal shapes (out-of-plane)
	High sensitivity accelerations	Accelerometers	At few nodal points (both planes)	Cost-effectively reduce entire noise level
Pseudo-static properties	Structural strain	Strain	Element under both tension and compression (L4-U5)	Measures structural strain, tests suitability of magnetic strain gage for structural elements

8.5 Campaign monitoring

This dissertation validates that using this WSS system, information from the input loads (train loads) and strain responses of chosen elements can be effectively measured. Strain gages can be installed at the rail level to estimate amplitudes of axle loads (and train speeds) for any train at any given speed. Magnetic strain gages were tested at the rail, but the results indicate that further development is needed for accurate load estimations. Magnetic strain gages can successfully collect bridges responses with high accuracy.

8.5.1 Track strain

To estimate train speeds, sensors were installed on the rail at two different locations on the bridge (Figure 8.5(a)). To estimate the speed and loads of the wheels, shear strains were collected in between the ties (Figure 8.5(b)). Conventional tee-rosette strain gages from Micro Measurements measured strain shear in the 62 kg. (136 lbs.) rail. The strains were measured at the centerline of the rail at 45 degrees (Figure 8.5(c)). The changes in strain under trains were sensed through a half Wheatstone bridge connected to the SHM-S strain sensor board. The strain collected was transmitted wirelessly to the gateway node, connected to a regular laptop during campaign monitoring. Figure 8.6 shows train speed estimation using the results from strain measurements at the two locations. The peaks correspond to each of the axles of an Amtrak train crossing the bridge at 31 meters /1.05 sec = 29.52 m/sec = 106 km/h (66 MPH). The difference in amplitude between the strain sensors corresponds to different boundary conditions and different sensor installation angle at the two different locations, as verified by a FE model of the rail developed for validation.

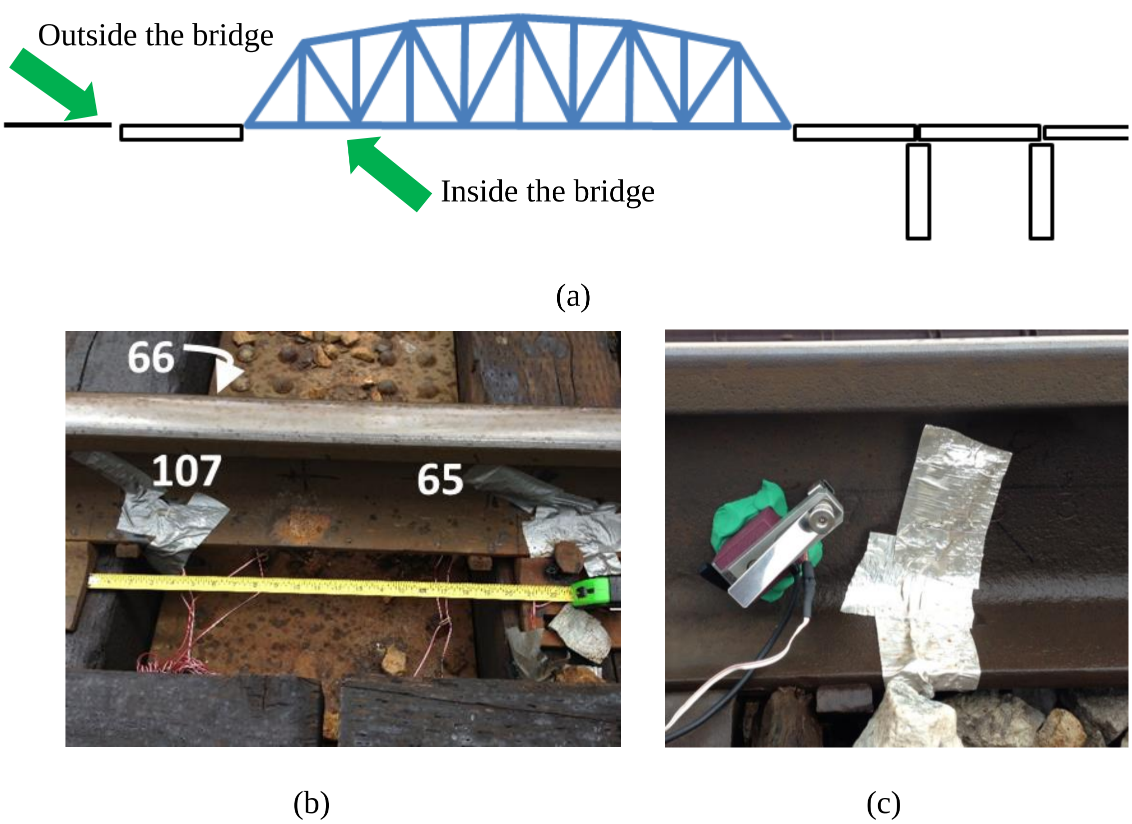


Figure 8.5 Rail strain sensors; (a) strain sensors location, (b) inside bridge, (c) outside bridge.

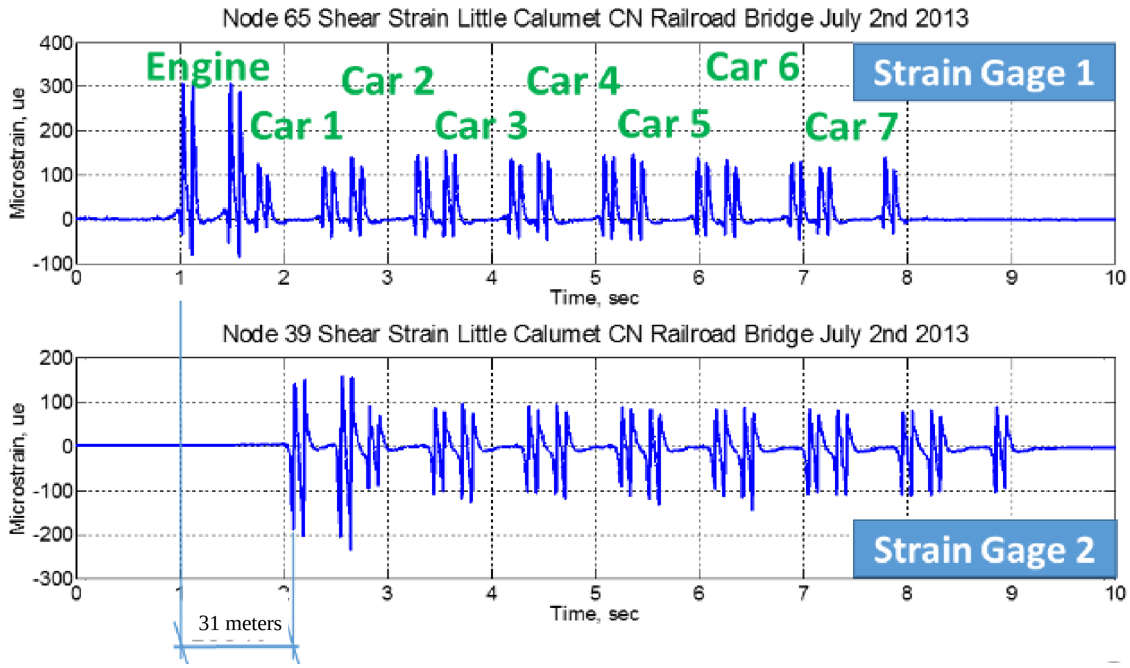


Figure 8.6 Estimation of train speed using two wireless strain gages installed in the rail.

The measurements in Figure 8.6 can assist to identify the type of car even prior to calibration. The first four peaks correspond to the four axles from the engine locomotive. The subsequent peaks (28 total) correspond to the seven cars of an Amtrak train. Because Amtrak is a passenger train, the difference in weight between locomotives and cars are large. However, difference in weight between locomotives and fully loaded freight cars are sometimes small, and calibration is required to distinguish specific weights from specific cars. The amplitudes of the rail strain were calibrated to measure the vertical load of the wheel crossing at each of the events. The shear stress between the two ties (spaced 50 cm (20 inches)), and the rail section properties were used to estimate the vertical load in the rail as a function of the strain:

$$f_v = \frac{V \cdot Q}{I \cdot t} \tag{8.1}$$

, where

V = total shear force in section, with $V = P/2$,

P = vertical load,

Q = static moment,

I = moment of inertia,

t = thickness where the stress is computed,

$f_v = \varepsilon \cdot G$, and

G = shear modulus.

Then,

$P = 0.1 \cdot \mu\varepsilon$.

To test the ability to predict wheel loads using WSS, estimated loads were compared with real loads from revenue service traffic. Train manifest is a document listing the geometry and weight of a given train. Norfolk Southern Corporation (NS) provided the train manifest for validation. The wheel loads were estimated and compared with the wheel loads provided by NS. Figure 8.7 shows that wheel loads estimated from the measured strains match well with actual wheel loads.

This research found that magnetic strain gages can be a potential tool to measure shear strain at the rail for quick wheel load estimation under revenue service traffic. Shear strain was collected under an Amtrak train using both traditional and magnetic strain gages (see Figure 8.8). Magnetic strain gages can estimate vertical wheel loads using WSS effectively and quickly, but the accuracy shows errors in amplitude. These errors are probably associated to the high level of vibration of the rail during train crossing events. However, using magnetic strain gages can be installed within seconds, as opposed to regular strain gages that require surface treatment. Railroad environments limit the time at the track for safety concerns. These preliminary results show that magnetic strain gages can be effective tools to obtain input loads to railroad infrastructures. However additional research is required to improve the accuracy of the strain amplitude using magnetic strain gages under revenue service traffic.

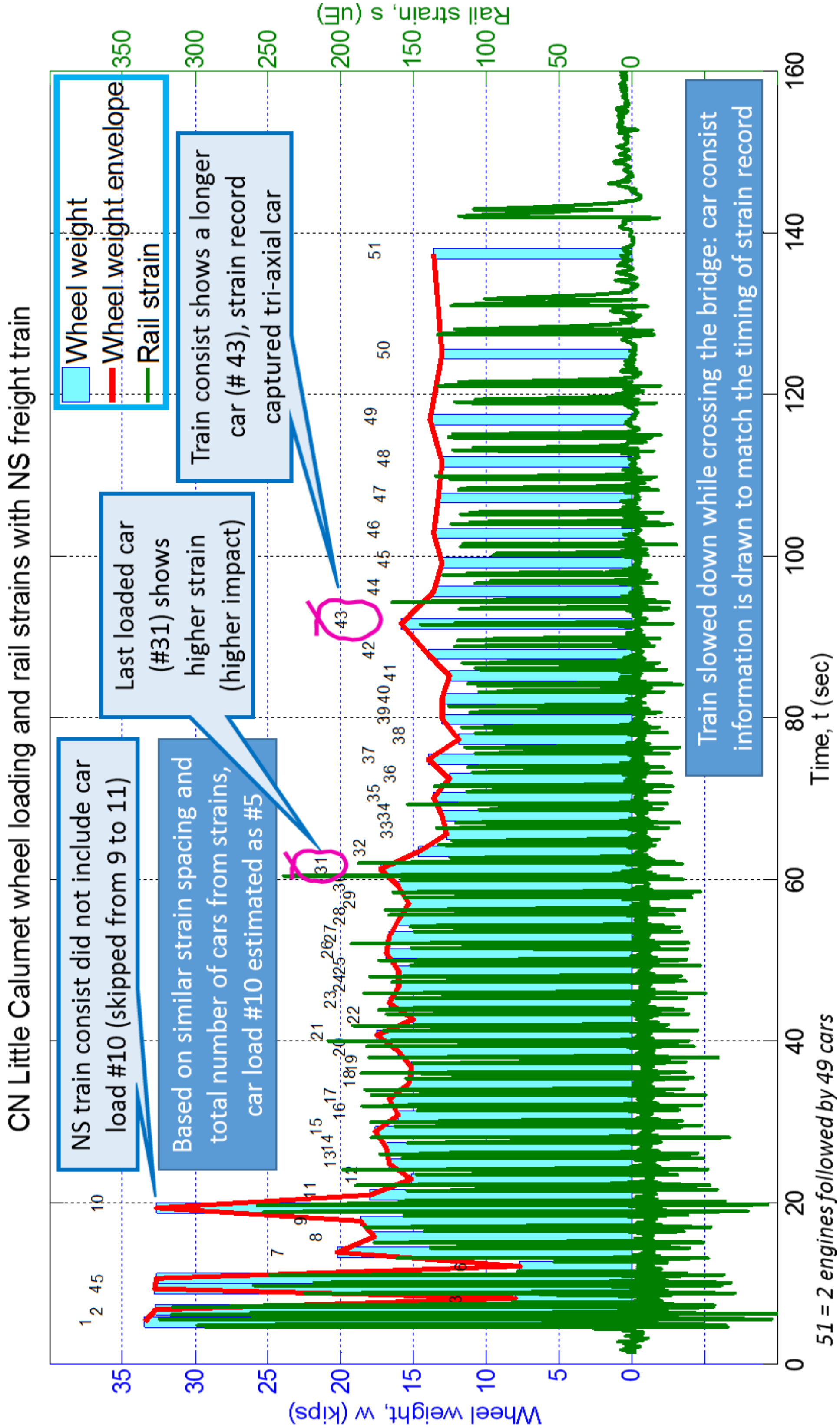


Figure 8.7 Estimation of car loading using wireless smart sensors.

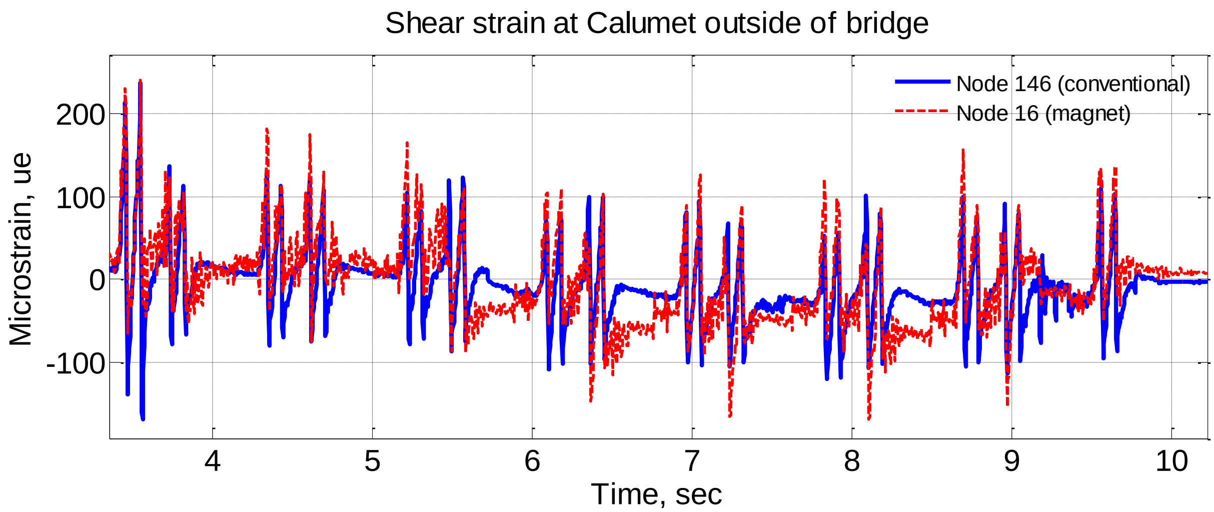


Figure 8.8 Comparison between conventional and magnetic strains at rail.

8.5.2 Structural strain

This dissertation used readings under a scheduled work train traffic of known speeds and loads provided by the railroad for this experiment. Figure 8.9 shows the loading of the work train used for this research. The work train crossed the bridge in both directions and in different speeds, ranging from 8 km/h to 74 km/h.

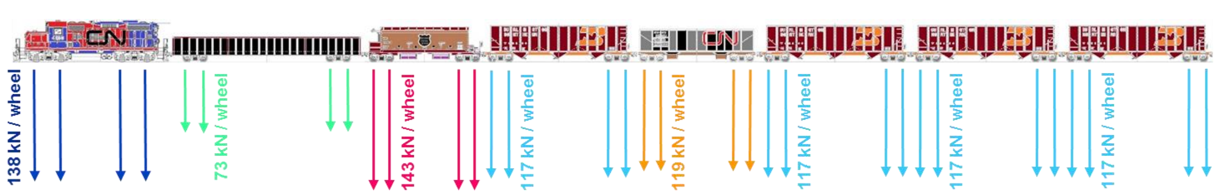


Figure 8.9 Work train wheel loading scheme.

The structural strain collected at the L4-U5 element can assess bridge response under train loading. The element L4-U5 is one of only two elements in the truss undergoing both tension and compression under train crossing events. Figure 8.10 shows both compression and tension under the different crossing locations during the event of work train crossing in the South Bound (SB) direction.

Figure 8.10 shows the strain measurements at both the structural elements and the rail under one of the work trains experiments (work train NB at 74 km/h (46 mph)). In order to validate the use of magnetic strain gages for campaign monitoring, the readings between both conventional and magnetic strain gages were compared. The upper figure compares both conventional and magnetic strain measurements at the structural element. The results show that they are nearly identical. Amplitude accuracy of a magnetic strain gages is better for applications estimating structural strain than applications estimating rail strain because the impact is lower at the structural element than at the rail level. Consequently, magnetic strain gages can be used for campaign monitoring of railroad bridges structural strains effectively. Inspectors can easily install magnetic strain gages at different points that they can choose while at the bridge in seconds with the same accuracy than traditional strain gages.

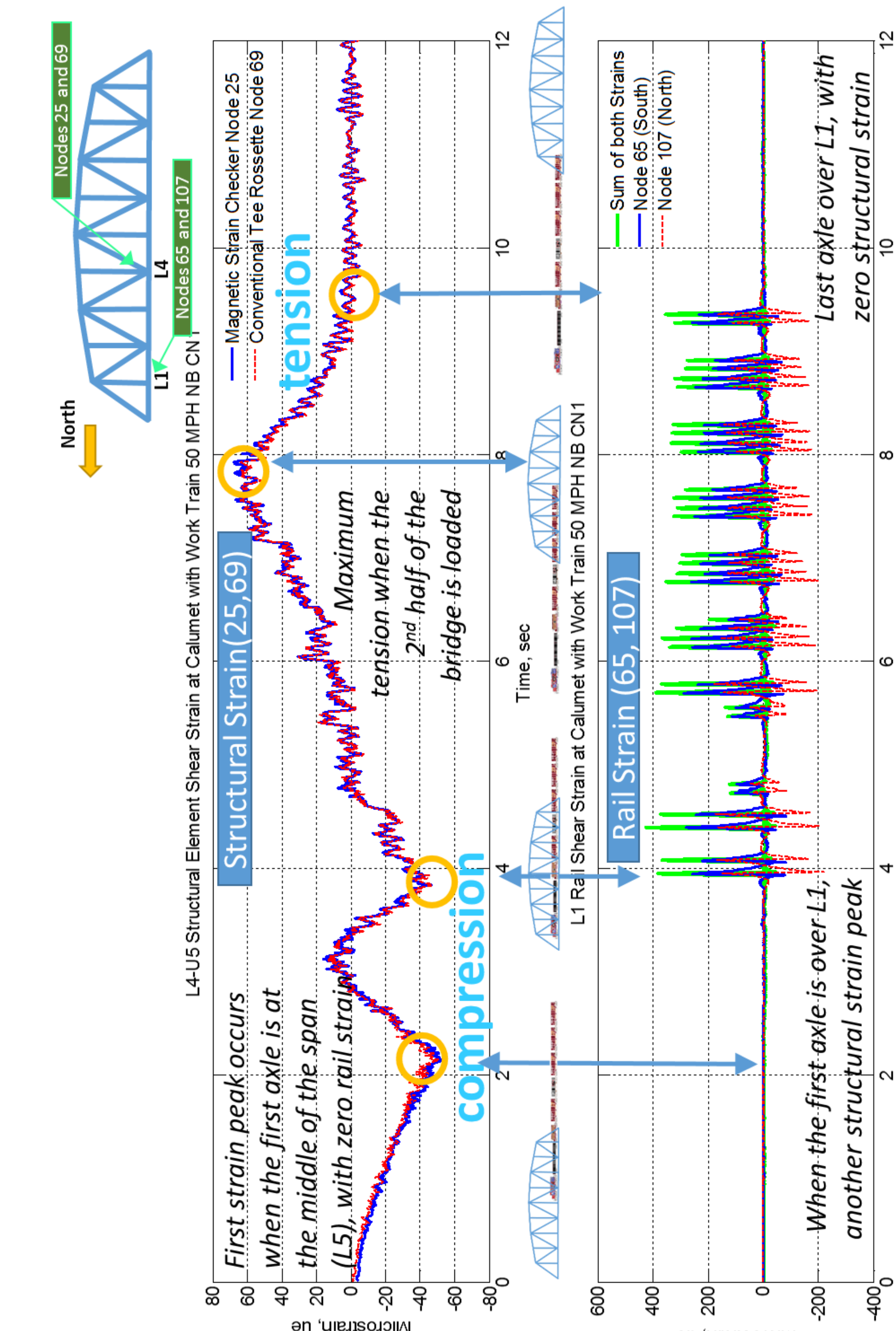


Figure 8.10 Strain measurement under multiple locations under work train.

8.5.3 Impact Factor

The strain collected under trains during campaign monitoring was used to estimate the impact factor (IF). This dissertation shows that the IF measured in the field is smaller than the design IF estimated in AREMA (2014). The AREMA manual determines the IF for steel railroad bridges as a sum of two effects: vehicle rocking (*RE*), and the vertical effects due to superstructure-vehicle interaction (*IV*), therefore *IF=RE+IV*. For truss spans and steam locomotives with hammer blow, the percentage of live load shall be (prior to *IF* reduction from AREMA for rating that is small):

$$IF = 15 + \frac{4000}{(L + 25)} \tag{8.2}$$

, where L = span length in feet.

Then,

$$IF = 15 + \frac{4000}{(310.42 + 25)} = 26.92\% \tag{8.3}$$

Strain measurements under trains running at various speeds during the work train experiment were similar in amplitude. Figure 8.11 shows that the strain measurements at diagonal member L4-U5 under trains running at different speeds in the NB direction are almost identical, with similar results under trains running in the SB direction. Table 8.2 summarizes the different IF estimated from the strain readings, which are smaller than the theoretical value estimated from the AREMA equation. The reason for lower IF in the structural strain is consistent with railroad empirical design approach, and indicates an IF for this double track at about 10% in average. The changes in the dynamic strain levels for these speeds are relatively small in comparison to the pseudo-static strain levels, and are independent of the speeds of this experiment.

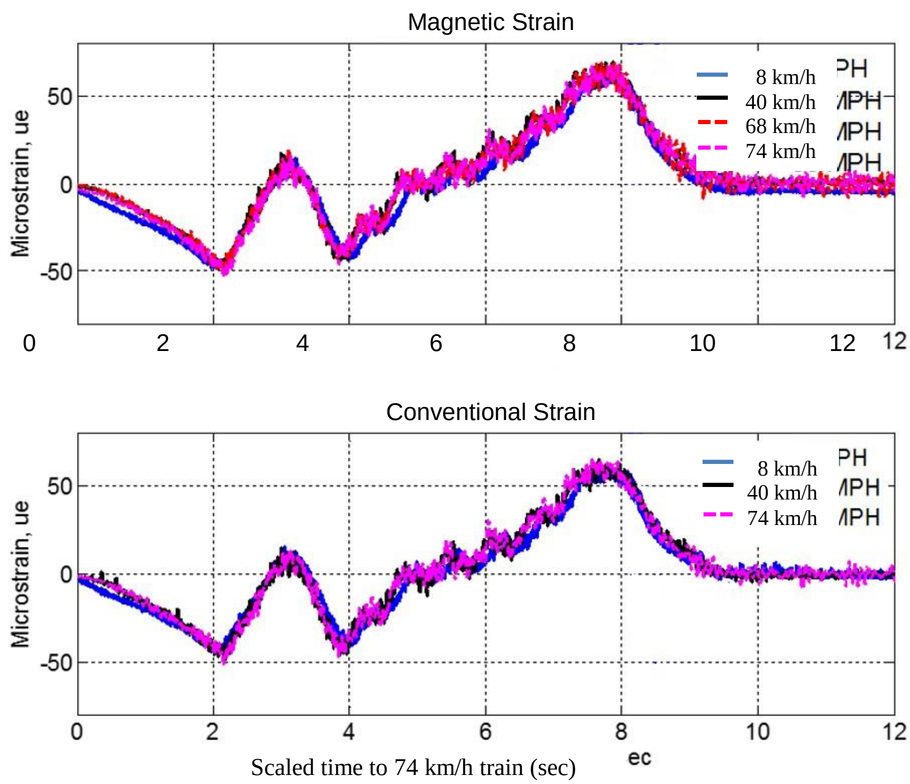


Figure 8.11 Magnetic and conventional strain of L4-U5 diagonal truss under different train speeds.

Table 8.2 Impact Factor estimation from rail shear strain at different speeds (NB).

Speed, s [km/h]	**Magnetic Strain**		**Conventional Strain**	
	Strain [με]	**IF**	**Strain [με]**	**IF**
8	61.48	NA	59.68	NA
40	69.48	13%	64.98	9%
68	68.74	12%	NA	NA
74	67.72	10%	64.74	8%

8.6 Remote autonomous monitoring

This dissertation validates a framework for monitoring railroad bridges using WSS. This system permits autonomous monitoring of input loads and bridge responses under revenue service traffic.

The system includes: (a) base station that reports autonomously and remotely to researchers over the internet; (b) autonomous monitoring framework that collects input loads and train axle information; (c) calibrated FE model, using both accelerations and strains; (d) strain estimation using the FE model; (e) autonomous strain estimation of measured strains using the FE model; and (f) ability to autonomously estimate strains of any element under any train.

8.6.1 Remote autonomous monitoring

Continuous remote monitoring during the duration of the project was achieved using a base station PC permanently installed by the bridge. This PC collected data wirelessly from the WSS. With a cellular internet connection at the bridge, the data was accessed remotely. The PC features include an Intel Atom N2600 1.6GHz processor with 4GB DDR3 Memory, with operating temperatures from -20^0 to 50^0 Celsius (- 4^0 to +140^0 Fahrenheit). The base station can collect the responses of the bridge under regular traffic continuously, and remotely. The data collected remotely added value for those applications where the railroad owners would like to install the system and let it collect data for multiple readings (days, weeks, months) without being at the bridge. This application provides safer monitoring because the data can be collected remotely.

8.6.2 Autonomous monitoring

Auto-monitoring using permanent deployed sensors can collect both train loads and speeds, as well as structural strain, remotely. When a certain acceleration was exceeded at the rail level, sensors woke up and collected data. During the period of August 26th and September 27th, over 30 remote sensing events were automatically initiated and stored. The autonomous monitoring measured both trains input loads and bridge responses. Figure 12 shows the rail strain collected remotely on October 24th (intermodal train). Green triangles in Figure 8.12(a) indicates the separation of the wheels, and the strain amplitude estimates the input load under each wheel. Figure 8.12(b) shows the structural strain collected with auto-monitoring at L4-U5 diagonal member. This application can be used to validate the predictability of the strains under multiple loading events.

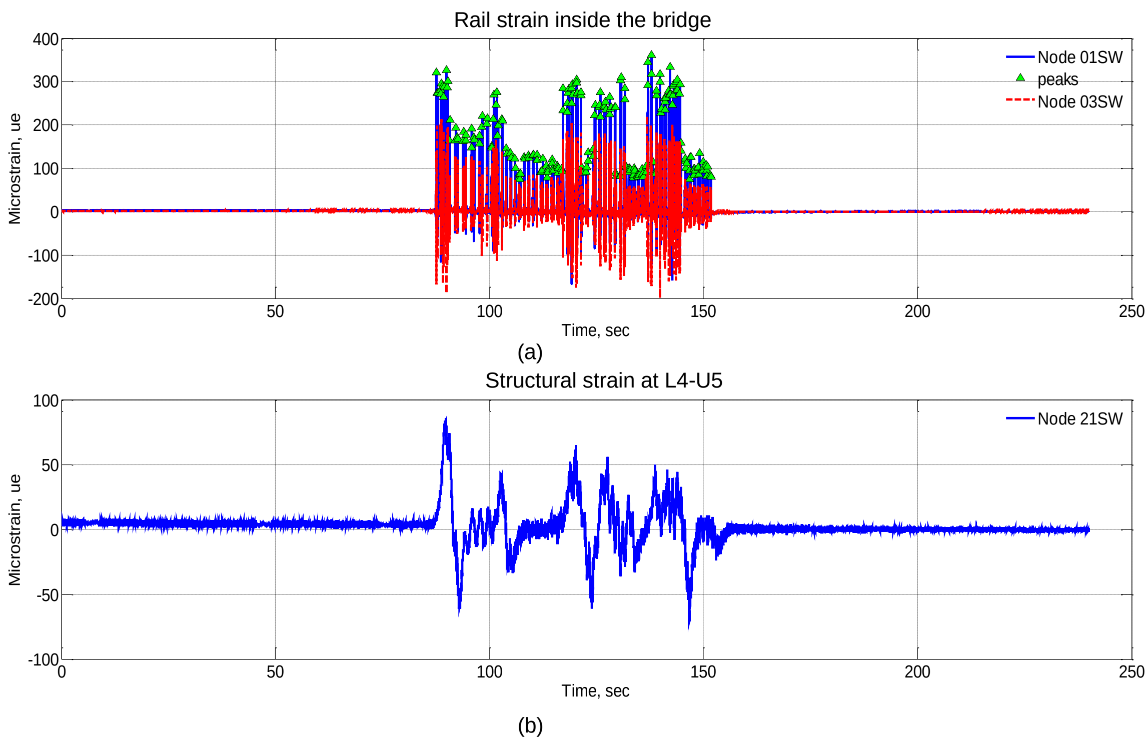


Figure 8.12 Strain collected during autonomous monitoring: (a) rail strain, (b) structural strain.

8.6.3 Calibrated FE model

The calibrated three-dimensional (3D) FE of the bridge represents the real bridge condition of the structure and it is built using both documents provided by the railroad. The FE model was developed in Matlab® (Figure 8.13(a)) based on the original construction drawings and used to pinpoint the location of sensors. The model contains 724 elements with sections properties extracted from the CN shop drawings. The floor system of the bridge is rigid; lower chords, floor beam, stringers and bottom lateral bracings form the floor system. The rigidity in the FE model is increased by calculating the moment of inertia about the reference axis, which is at the center of lower chords (Figure 8.13(b)). To calculate stresses in the model, researchers used the gross and net areas of the members according to the AREMA Manual (2014) (Figure 8.13(c), Figure 8.13(d), and Figure 8.13(e)). The results obtained using the FE model can be compared to the measurements to illustrate the potential of estimating strains under revenue service traffic using WSS.

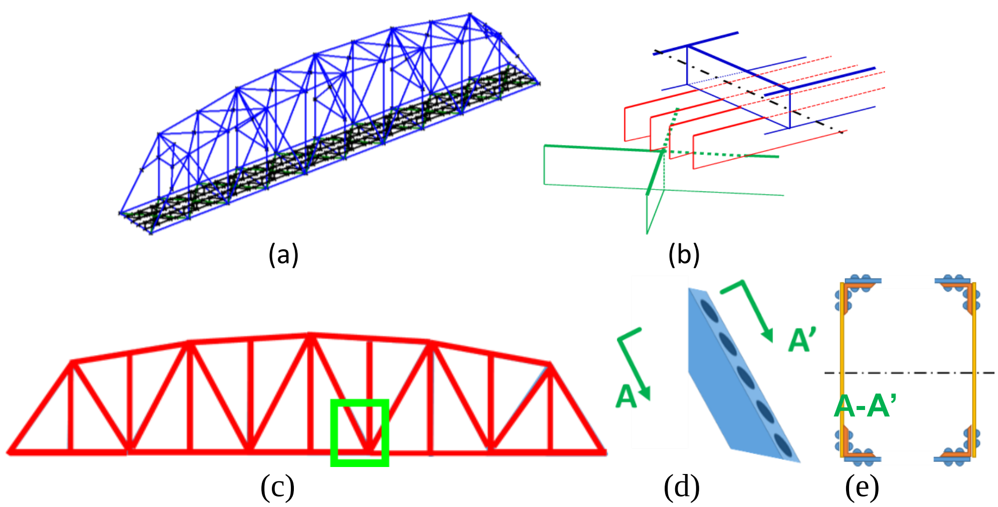


Figure 8.13. FE model: (a) 3D view, (b) floor system, (c) elevation, (d) element, and (e) gross area.

8.6.4 Strain estimation on the FE model

Strain measured matched strain estimated by the 3D calibrated FE model. Figure 8.14 shows the comparison between the measured strain and the predicted strain from the FE model. Using static analyses, the FE model matches well the measured strain. These results demonstrate the predictive capabilities of the FE model, providing a good tool for understanding the behavior of the bridge under revenue service traffic.

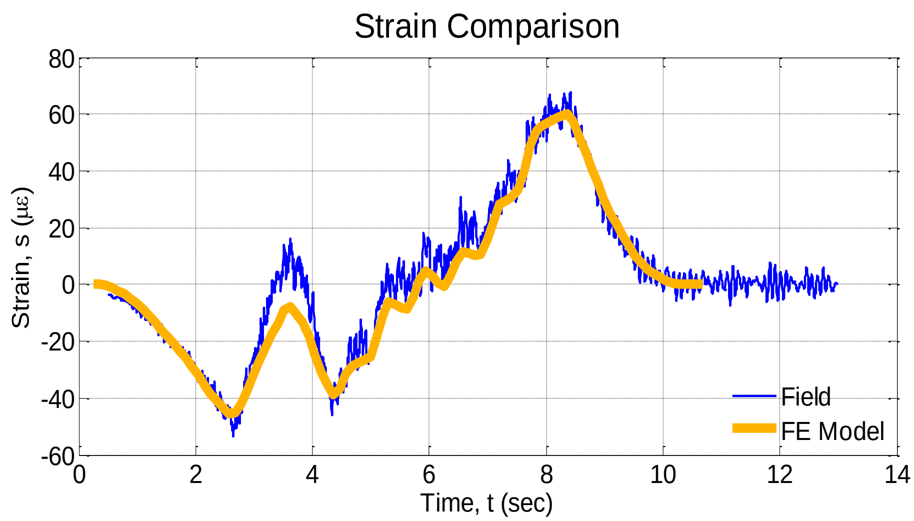


Figure 8.14. Strain prediction for L4-U5 diagonal member using FE model.

8.6.5 Autonomous strain estimation

Strains from the 3D FE model for the different traffic conditions and direction closely match the strains measured at the Calumet Bridge for the L4-U5 diagonal member. Figure 8.15 shows the comparison between predicted and measured strains under a SB train crossing the bridge at 54 km/h (33 mph). Auto-monitoring can collect the input loads and speeds from the rail and use them as an input for the FE model, which effectively estimates the strain.

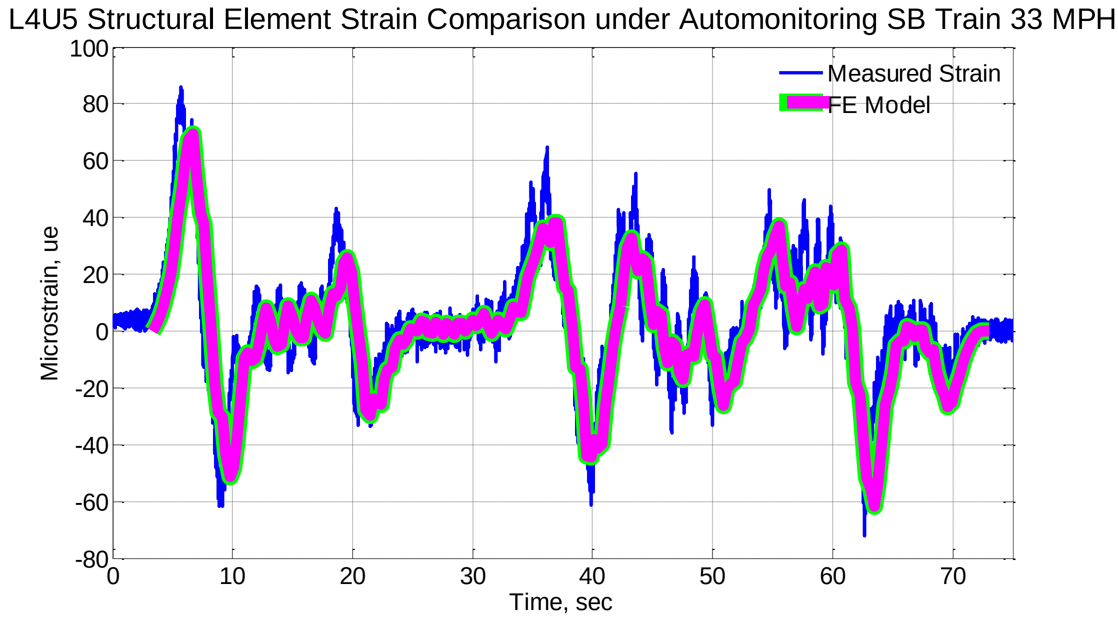
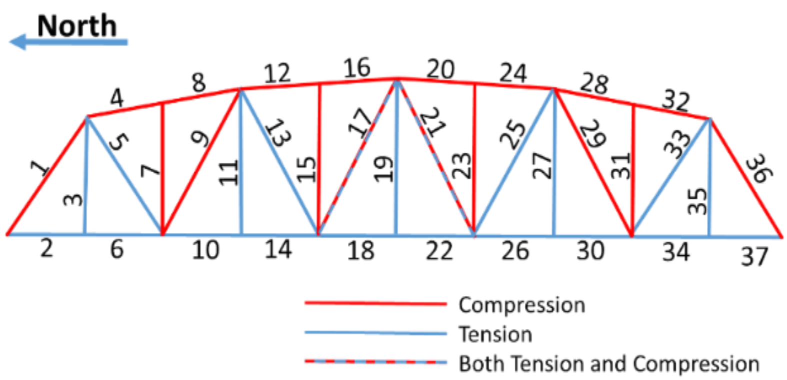


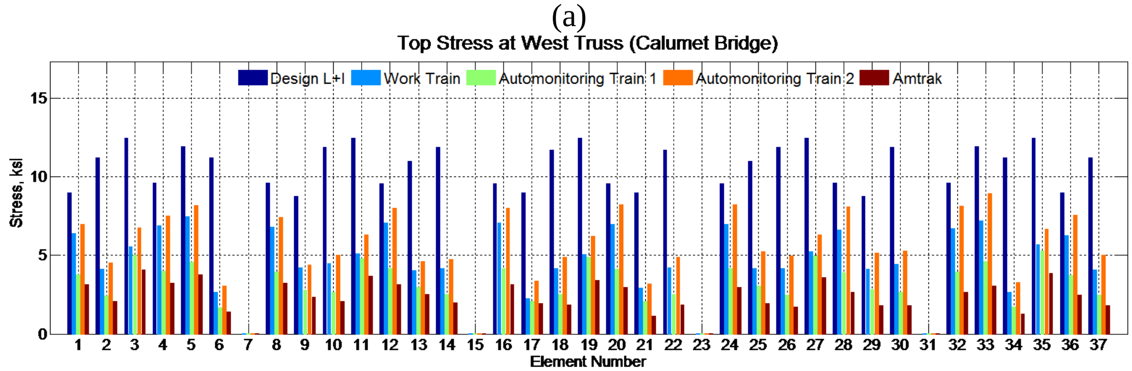
Figure 8.15. Auto-monitoring strain validation under SB train.

8.6.6 Autonomous strain estimation of any strain under any train

The 3D calibrated FE model can predict the strain of all the members under trains. Figure 8.16(a) shows all of the members of the West truss plane of the bridge. Figure 8.16(b) shows results of autonomous monitoring of four different trains crossing the bridge at four different times, remotely. Using the input loads obtained by remote monitoring, the FE model determines the maximum strain of each of the 37 members in each of the trusses (Figure 8.16(b)) under each different event. From the four events in Figure 8.16(b), Train 2 causes the maximum strain, followed by the Work Train, Auto-Monitoring Train 1, and Amtrak, respectively. In all four cases the strain levels are under the design strain levels per the design drawings. The design stress level was computed using live + impact loads (L+I). Because these strain levels are under one train in the West track, two trains will cause higher strains. Vertical posts (elements 7, 15, 23, and 31 in Figure 8.16(a)) have zero stress, whereas top chords (elements 4, 8, 12, 16, 20, 24, 28, and 32) and end posts (elements 1 and 36) have higher stresses. The estimated strains for all the members under revenue service traffic can be used for continuously, remote, and safe monitoring of the bridge.



(a)



(b)

Figure 8.16. Predicted stresses under open regular traffic levels at West truss (Calumet Bridge).

8.7 Conclusions

This chapter provided a new WSS framework application to monitor railroad bridges responses under trains using WSS. The WSS monitoring system was deployed on a CN bridge on the South side of Chicago, Illinois. This new field application shows that these sensors and data collection strategies are appropriate for harsh railroad environment and conditions. Wireless strain gages are used to measure real-time train loads. Wheel loads, spacing, and speeds were determined from two strain sensors placed on the rail. Bridge responses were measured under revenue service traffic (both freight trains and Amtrak), including accelerations, structural strain, and rail strain. Researchers verified the applicability of a magnetic strain gage for quick deployment and measurement of strains in structural elements. Results show that strain measurements at the rail using magnetic strain gages needs further research under revenue service traffic. Measured data demonstrated that the primary response of the bridge was pseudo-static. The IF for the bridge under multiple experiments was between 8 and 13%, always less than 50% of the design IF by AREMA. Subsequently, the maximum stresses/strains at arbitrary locations on the bridge are determined primarily from the pseudo-static response of the calibrated FE model. Using the calibrated FE model, and the input data, the WSS system estimated the strain for all the elements of the bridge under revenue service traffic. A strain map of the structural elements in the bridge under any given train loading can be obtained. A cellular internet connection on the base station enabled an autonomous notification service to the researchers so data was collected remotely, without personal at the bridge. By having a long-term WSS monitoring deployment, the WSS framework can measure responses under different trains automatically, continuously, inexpensively, and safely. Railroads can then compare changes in responses between annual bridge inspections, and in the case of steel bridges, predict remaining fatigue life. Once a specific bridge assessment is complete from a normalized measurement under one train, inspectors can quickly compare these sets of data with past responses of the same element, or even different elements. This chapter validated that wireless sensors can be an effective and accurate measuring tool for monitoring railroad bridges performance. Results shown in this chapter provided examples on how wireless sensors can inexpensively and safely collect quantified information such as strain of steel railroad bridges under measured input loads. Using measured loads remotely, a FE calibrated model can predict strains in all members of the truss. The proposed system can be adopted by the consequence-based framework to assist informed decisions for the prioritization of repair/replacements of bridge elements.

CHAPTER 9 CONCLUSIONS AND FUTURE STUDIES

9.1 Conclusions

This dissertation proposes an initial framework for the consequence-based management of railroad bridges for making network-wide MRR decisions. Because the operational costs are uncertain, the goal established here is to use objective information to inform expected operational costs. This framework minimizes the expected value of the total network cost. The proposed framework employs fragility curves to this end, which relate service condition limit-states to bridge displacement under revenue service traffic. The operational costs associated with these service conditions can be used to estimate the total costs of a given MRR policy. In this way, this framework proposes intelligent MRR decisions that minimize the total network costs to railroad operations. Additionally, measured bridge data can be used to update periodically the fragilities to have more accurate estimates of the bridge condition. Using this framework the rail owner can identify the most efficient use of a limited budget while maintaining safe railroad operations. This dissertation has additionally used WSS to be a practical, efficient, and robust means to collect information that can be used to inform MRR decisions based on objective information. The applications focused in this dissertation were the monitoring of railroad revenue service traffic loads and responses using simplified monitoring, and the estimation of reference-free displacements. With these applications being inexpensive, effective, and reliable, railroads can measure bridge responses in almost real-time to describe the bridge condition based on quantified bridge responses under revenue service traffic. Field experiments at several railroad bridges were performed to support the value of using WSS in the railroad bridge environment. The main benefit of this new tool was the prioritization of railroad bridge networks MRR decisions based on objective data.

9.2 Future studies

9.2.1 Framework validation for SHM of infrastructure

The results obtained for the specific railroad bridge network need to be expanded for larger railroad bridge networks. A larger pool of data obtained with simplified monitoring can provide better evidence between the relationship between service limit states and bridge responses under revenue traffic. In particular, areas of improvement of the network are:

1. This method was developed for a bridge type, but it could be expanded to incorporate all types of bridges in the network, incorporating their fragilities to make decisions of the entire network of bridges.
2. This study was directed to railroad bridges but the framework could be applied to other transportation infrastructure that is subjected to large demands by their current day-to-day loading condition. Highway bridges of specific types can benefit from this approach using simplified data collected that can be used to inform highway bridge authorities about the current bridge condition based on their response to loads.
3. The framework proposed herein can be expanded to critical components of the power grid, such as wind turbines, solar farms, or water reservoirs, subjected to high performance demands.

The ultimate goal of this future work is to provide consequence-based data sensing, processing, and regulations for safe, sustainable, and cost-effective management of civil infrastructure (Figure 9.1).

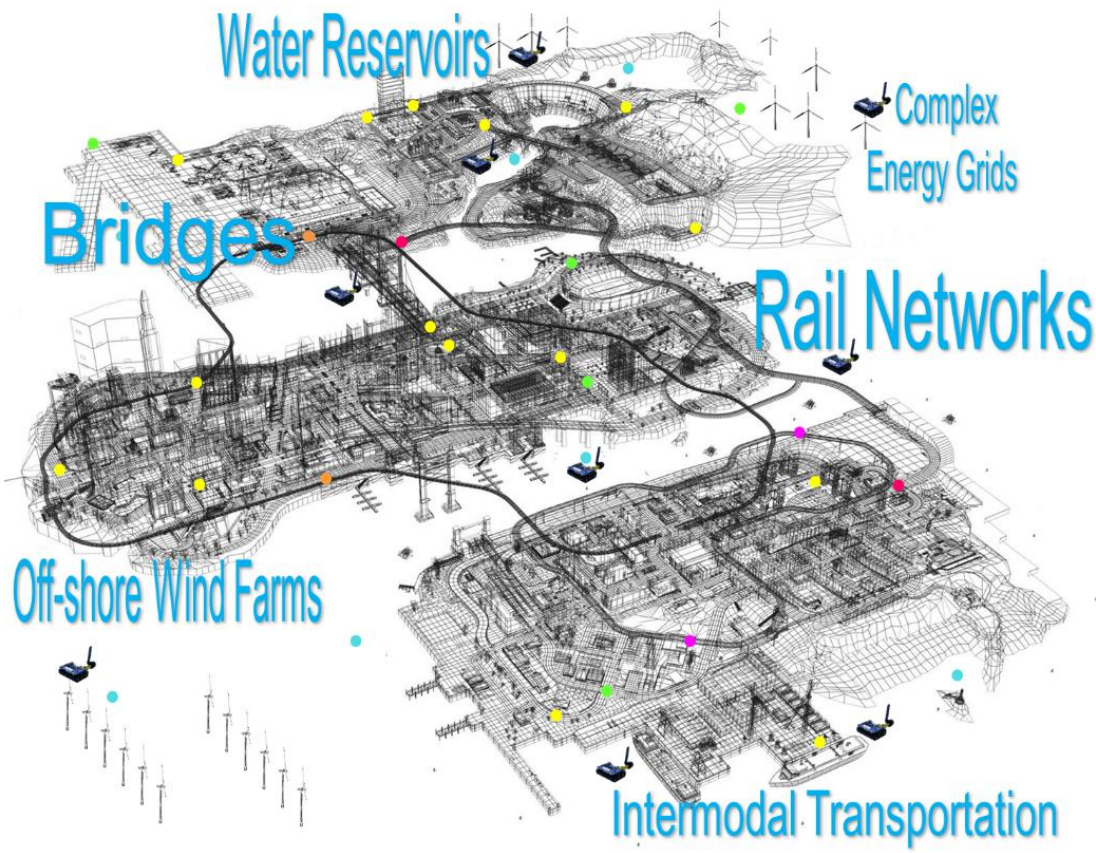


Figure 9.1. Consequence-based (Performance) Monitoring of Civil Infrastructure.

9.2.2 Sensing development for railroad bridges monitoring

Another area of research that is motivated from this research is the development of a new generation of WSS designed for the monitoring of railroad bridges. During the course of the WSS monitoring it was found that the harsh environment around railroad bridges requires specific applications. These developments include, but are not limited to:

1. Sensors (hardware): ability to record higher acceleration amplitudes under trains. Current WSS are designed for ambient vibration, and the accelerations under trains require larger orders of magnitude.
2. Sensors (applications): new sensing capabilities that can assist measuring displacements using multi-metic sensing. These sensors include tiltmeters or strains that can measure timber strain under trains.
3. Sensing strategy (applications): railroad bridge impact monitoring for under passing traffic, including, but not limited to, highway traffic and navigable vehicles.
4. Signal (communication) development: antenna signals have limited range and due to limited access to railroad bridges need to be developed for longer distances. Development of antennas with longer range can assist to monitor railroad bridges that are usually non accessible for railroad bridge inspectors.
5. Magnetic strain sensor (applications): rail strain measurement with friction type devices without surface treatment are of interest for monitoring railroad bridges. This application needs to be explore to obtain accurate strain measurements that can assist measuring of train loads during monitoring campaigns.
6. Sensor enclosure: this dissertation has measured both timber and steel railroad bridges using the magnetic application of WSS. However, new developments in new materials can assist to develop new enclosures specifically developed for the three materials composing the population of railroad bridges: steel, concrete, and timber.
7. Railroad bridge performance limits. Using evidence of transverse displacements of changes of bridge serviceability can assist to determine and include limit(s) on transverse

displacements in their assessment practice and/or the AREMA manual, in addition to the current AREMA limit on normalized vertical displacements under trains.

8. Railroad bridge and track performance assessment using simultaneous sensing under traffic, toward identifying track-bridge responses relationships and possibly new monitoring methods that inform safety of revenue service traffic operations.

9.2.3 Multidisciplinary approach to infrastructure monitoring

The consequence-based framework can potentially serve multiple industries, while the example and development presented in this dissertation has been directed to the application of the framework in the context of railroad bridge networks maintenance prioritization. Multidisciplinary developments of the framework can provide a robust assessment of large and distributed infrastructure elements based on normalized, objective information about their performance. The holistic development of each application requires contacting the owners or stakeholders familiar with the most pressing issues and providing the information that is identified of value for their decision-making. In order to develop the power of the framework the following improvements are suggested:

1. Assessment of the uncertainty between service limits and the bridge response based on complete information of the data collected including, but not limited, to: sensing precisions, bridge representativeness within the network, representativeness of the bridge responses of the health of the bridge, and relevance of the bridge performance under trains to the service limit state of the bridge.

2. Development of WSS algorithms particularly suited to reference-free displacement estimation that can reduce the error limits, including, but not limited to, data fusion, multimetric sensing, and vision identification.
3. Unmanned Aerial Vehicle (UAV) development for sensing of infrastructure. Development of new strategies for monitoring infrastructure remotely using UAVs that can inform owners about the health of the bridge accurately and inexpensively.
4. Engineering description of current infrastructures assessment and code development to implement and develop new regulations that use objective data about infrastructure using inexpensive monitoring resources to increase safety and reduce costs.

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