MCEER
EARTHQUAKE ENGINEERING TO EXTREME EVENTS

MCEER Special Report Series on Multiple Hazard Bridge Design

U.S. Department of Transportation
Federal Highway Administration

Design of Highway Bridges Against Extreme Hazard Events: Issues, Principles and Approaches

Edited by

George C. Lee, Mai Tong and W. Phillip Yen

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MCEER is a national center of excellence dedicated to establishing disaster-resilient communities through the application of multidisciplinary, multi-hazard research. Headquartered at the University at Buffalo, State University of New York, the Center was originally established by the National Science Foundation (NSF) in 1986, as the National Center for Earthquake Engineering Research (NCEER).

Comprising a consortium of researchers from numerous disciplines and institutions throughout the United States, the Center's mission has expanded from its original focus on earthquake engineering to address a variety of other hazards, both natural and man-made, and their impact on critical infrastructure and facilities. The Center's goal is to reduce losses through research and the application of advanced technologies that improve engineering, pre-event planning and post-event recovery strategies. Toward this end, the Center coordinates a nationwide program of multidisciplinary team research, education and outreach activities.

Funded principally by NSF, the State of New York and the Federal Highway Administration (FHWA), the Center derives additional support from the Department of Homeland Security (DHS)/Federal Emergency Management Agency (FEMA), other state governments, academic institutions, foreign governments and private industry.

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Preface

George C. Lee, Mai Tong and W. Phillip Yen

After visiting areas affected by major natural and manmade disasters during the past several years in the U.S. and abroad, the authors of this report identified a need to consider the development of multi-hazard design principles and guidelines for highway bridges. They began by reviewing existing information with the aim to expand the methods and approaches established in earthquake engineering. During the past two years, a number of exploratory studies have been carried out to define a potential research project that may be accomplished within a period of several years with a moderate but reasonable budget. These pilot studies included the following:

1. Understand the physical conditions of existing bridges and current methods of bridge design, retrofit and inspection.
2. Identify current philosophical approaches to handle multi-hazard (extreme events) load effects on highway bridges.
3. Perform scenario studies of possible damage in a typical highway bridge model due to various extreme hazard loadings.
4. Explore various means to compare some extreme hazards and their load effects.
5. Examine the suitability of using the return period as the basis for comparing different hazards.
6. Conduct uncertainty analysis of extreme hazard models.

The studies revealed the complex nature of establishing multi-hazard design principles and guidelines for highway bridges, since there are so many interrelated parameters to be examined within many different contexts for consideration of importance and priorities. Furthermore, there is insufficient information on hazard occurrences and their load effects on the capacity of bridges, such as damage data, etc. In order to begin to establish a research agenda, the authors organized a special workshop attended by a variety of experts including experienced designers, stakeholders, code authors and researchers. The participants offered their views and opinions on key issues and principles in the multi-hazard design of highway bridges from their unique perspectives. Their papers comprise this report and the central thoughts of these contributions are summarized as follows:

Chapter 1: Importance of and need to pursue a study to establish multi-hazard design principles and guidelines.

Chapter 2: The view of an experienced bridge designer advocating that emphasis be given to simplicity, relevance and the need to consider risk from a uniform perspective.

Chapter 3: The view of an owner and stakeholder that uniform risk-based considerations of extreme events need to be considered. However, mitigation approaches must be developed within the context of normal loadings and functions of bridges.

Chapter 4: Based on two years of preliminary study, the authors present a case for developing a platform for evaluation, comparison and quantifying extreme hazard events as the first important step toward the establishment of multi-hazard design principles.

Chapter 5: An approach is proposed to first establish a probability-based seismic hazard table for decision-making, which adopts a design principle by comparing performance with risk, and then expand it to include other extreme events. The table would contain information such as the probability of hazard level vs. vulnerability of bridge and performance (e.g., likely level of collapse).

Chapter 6: A case study is presented to illustrate the challenges in establishing design principles to predict uncertain future extreme hazard load effects for which only limited historical data is available.

Chapter 7: This chapter provides an illustrative example on developing simplified design guidelines based on extensive research efforts on LRFD-based earthquake resistant design of highway bridges.

Chapter 8: This chapter articulates a need to establish a comprehensive bridge failure database on all types of hazards, based on the experience and conclusions of the preliminary studies.

Chapter 9: The summary outlines a possible roadmap based on the results of the preliminary studies and views of the experts expressed in this volume.

While design against multiple hazards is a complex issue, it is only a small component in the total landscape of bridge engineering design practice, which has well-established codes and specifications based on years of experience and scientifically-based studies. Many concerns and key issues have been raised in the papers presented herein. However, this report should not be viewed as a text that provides answers on multi-hazard load effects to the bridge design community. Instead, its purpose is to provide valuable input to the research community to help plan and design a research program.

We plan to pursue the research path identified from the discussions and the expert advice received at the workshop, and are anticipating that this report will be the first in a series dedicated to the issues and challenges facing the bridge community in the design of bridges against multiple hazards. Future volumes will address this topic in greater detail, and will include a variety of contributors, such as members of AASHTO T-3, FHWA, MCEER, and other key professionals. “Progress reports” on MCEER research related to this subject will also be included.

Multiple Hazard Research Efforts With the AASHTO Code Development Activities

1

Harry A. Capers, Jr.

This chapter presents a number of challenging issues on multi-hazard research from the perspective of establishing the American Association of State Highway and Transportation Officials (AASHTO) design specifications including utilizing and understanding isolated past studies and the grand challenge to advance the current AASHTO specifications to better represent loads that new designs will experience during their service life.

AASHTO's Subcommittee on Bridges and Structures' (SCOBS) work on the Load and Resistance Factor (LRFD) specification began back in 1986. A group of bridge engineers wrote a letter of concern to the AASHTO Highway Subcommittee on Bridges and Structures expressing their concern over the ability of that body to continue to maintain the existing Standard Bridge Specifications, the first edition of which had been issued in 1931. As a result, a synthesis project was performed, NCHRP Project 12-28(7), "Development of Comprehensive Bridge Specifications and Commentary" which supported the concerns of the original letter submitted to the SCOBS and recommended a project to develop a calibrated, reliability based specification to replace the existing specification. The National Cooperative Highway Research Program undertook a project to address this problem, NCHRP Project 12-33 in July 1988, and the result was the adoption of the first edition of the new Load and Resistance Factor Specification in 1994 with several editions following over the years.

The prior AASHTO specifications were based on Working Stress and Load Factor design philosophies. These methods use some fraction or percentage of a given material's load carrying capacity and adjustments to design factors to reflect the variable predictability of certain load types, such as vehicular loads. In contrast, the LRFD specifications consider the variability in the behavior of structural elements. Adjustments to design factors to reflect the variable predictability of loads and materials is included in the specification and is a positive characteristic of the LRFD specification that did not exist in older specifications. Also, prior specifications did not provide distribution factors for vehicular loads as accurately as current information would allow. Finally, extensive use of statistical analyses to ascertain the behavioral variability was performed in developing the LRFD specifications. All of this leads to the belief that the LRFD specifications represent a major step forward in improved bridge design over past specifications and provides more accurate analysis methods, the use of which will lead to bridges that exhibit economy of design, superior serviceability and more uniform levels of safety.

The basic equation for LRFD is as follows:

$$\sum \eta_i \gamma_i Q_i \leq \phi \ R_n$$

where: Q_i = Load or force effects

R_n = Nominal resistance

η_i = A factor relating to ductility, redundancy and operational importance

γ_i = Load factor, a statistically based multiplier

ϕ = Resistance factor, a statistically based multiplier

Load and resistance factors developed for use with this formula have been calibrated by trial designs to provide a high and uniform level of safety in new bridges. This calibration provides that bridges designed in accordance with LRFD have a more uniform level of safety as is illustrated in Figure 1-1.

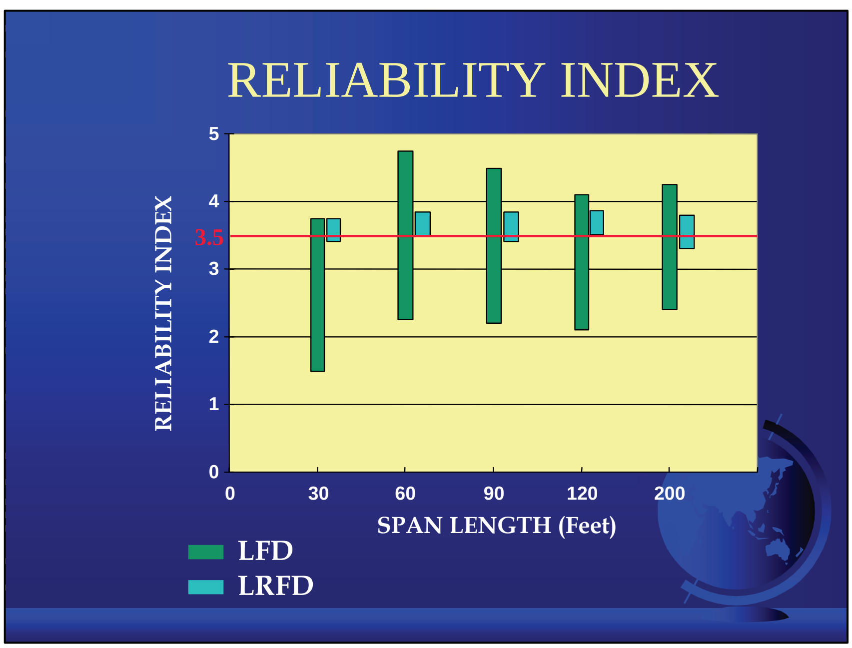


Figure 1-1. Comparison of Reliability Index for LRFD and LFD Specifications for Various Span Lengths

The safety level, expressed by a reliability index, β, provides a reliability index of approximately β = 3.5 for different types and configurations of bridges. This reliability index of β = 3.5 provides a 1 in 10,000 notional failure probability, or put another way, assures that only 1 out of 10,000 design elements or components will have the sum of the factored loads greater than the factored resistance during the design life of the structures.

Figure 1-1 also shows a comparison of the older AASHTO Standard Specifications. As can be seen, that specification did not provide for a consistent and uniform safety level but provided a reliability index as low as 2.0 or as high as 4.5. A β = 2.0 is a very high rate representing 4 out of 100 design elements and components that would probably be overloaded and experience a problem at some time during the design life of the structure, and would result in high maintenance needs or component failure. On the other hand, a β = 4.5 would result in a design that would be very conservative and costly.

The LRFD specification adopts a limit state philosophy or a state beyond which a component ceases to satisfy the provisions for which it was designed. The limit states provide a systematic approach to structural design to ensure satisfactory short- and long-term performance of bridges. LRFD defines four main limit states to be satisfied by design. These are:

1. *Strength Limit State*: This limit state stipulates the strength and stability requirements to resist the specified load combinations expected to be experienced by a bridge over its design life. This limit state assures adequate ultimate load capacity.
2. *Extreme Event Limit State*: This limit state deals with the performance of a bridge during earthquakes, scour or other hydraulic events, ice loads or ship collision.
3. *Service Limit State*: This limit state imposes restrictions on stress, deformation, and crack width under service conditions. This limit state assures elastic behavior and minimizes need for maintenance during the service life of the structures by addressing cracking and other conditions.
4. *Fatigue and Fracture Limit State*: This limit state imposes restrictions on stress range due to a design truck occurring at the number of expected stress cycles. Again, this is similar to the fatigue design requirements in Working Stress Design and Load Factor Design to assure that there is no premature fatigue cracking or failure in the members of the structure.

Unfortunately, due to the scope and costs associated with calibration of each of these limit states, to date only the strength limit state has been calibrated using reliability methods. The remaining limit states have only been “calibrated” to past practices in the older specifications.

These AASHTO Load Cases have been the “Comfort Zone” but may not represent the current state of the art or meet the public’s expectation of how our nations bridges should perform beyond the strength limit state. Recent history shows many examples of events occurring to the nation's transportation system, the magnitude of which far exceeds expectations assumed in the development of the code. Many of these events provide new data that must be considered in developing standards that result in designs that perform to expectations.

These events also challenge us to consider looking at how we assume bridges will be loaded. AASHTO load cases assume simultaneous application of loads that must be resisted. In reality, structures are seeing multiple and cascading events. Structures that have sustained damage from one event must resist additional loads inflicted by a subsequent event prior to an agency being able to make repairs to the initial damage sustained. Many other fields such as aviation and nuclear engineering address these types of events. The highway transportation industry needs to follow their lead.

Recent events have revealed that the public expects owners to act responsibly regarding their safety and react to threats to life to the best of their ability. However, it also shows that the public's expectation regarding post event recovery is reasonable in that they will tolerate deliberate, cost effective efforts that will bring facilities damaged in an extreme event back into service in a reasonable timeframe. If we accept these assumptions two question come to mind. First, what is the actual tolerance level that should be assumed? Second, what recent lessons learned and "best practices" are available that owners might immediately adopt to prepare for an event.

Recent events have also indicated that current design specifications and procedures do not represent the maximum hazard level that facilities see nor do they include all of the types of hazards that occur. Wave action and fires are not currently addressed in the current bridge specifications but have resulted in extreme damage to bridges in several regions of the state. While it is impractical to think that a design could be produced to deal with the worst hazard possible at a site we need to re-examine our approach to dealing with extreme events in our designs to insure life safety and viability of a region after a disaster.

The AASHTO SCOBS has sponsored quite a few projects with the goal of improving structural performance. These include examining structural redundancy, seismic analysis of bridge superstructures and substructures, scour and extreme events in general. A complete listing is available on the TRB website. Other efforts are underway as pooled fund studies or as state sponsored research projects to examine topics such as coastal scour, wave action and ship collision. Much security related projects have been initiated and completed in the post 9-11 era. Actions relating to fire hazard are expected to be a priority especially with the formation of a Technical Committee on Tunnels and development of Tunnel Design standards and as a concern of the Technical Committee on Security as well as the Technical Committee on Loads in view of recent major bridge losses.

How these loads are applied is equally a concern as more experience is gained in working with the new specification. In some instances using the current limit states has required substantial designs to meet the capacity demand requirements. It is an accepted fact that target hardening bridge to resist a deliberate attack is both difficult and costly. Also, all hazards are not applicable in all locations; bridges in some areas of the country are at higher risk to certain extreme events and not others. The question may be whether the limit state approach currently in the AASHTO design specification is the best approach or would a risk based approach allowing owners to specify performance standards acceptable for a site produce better designs. Several including MCEER are examining such an approach.

The AASHTO Subcommittee on Bridges and Structures has identified this as a priority. The following challenge was outlined in their June 2005 document titled: **Grand Challenges:** ***A Strategic Plan for Bridge Engineering***

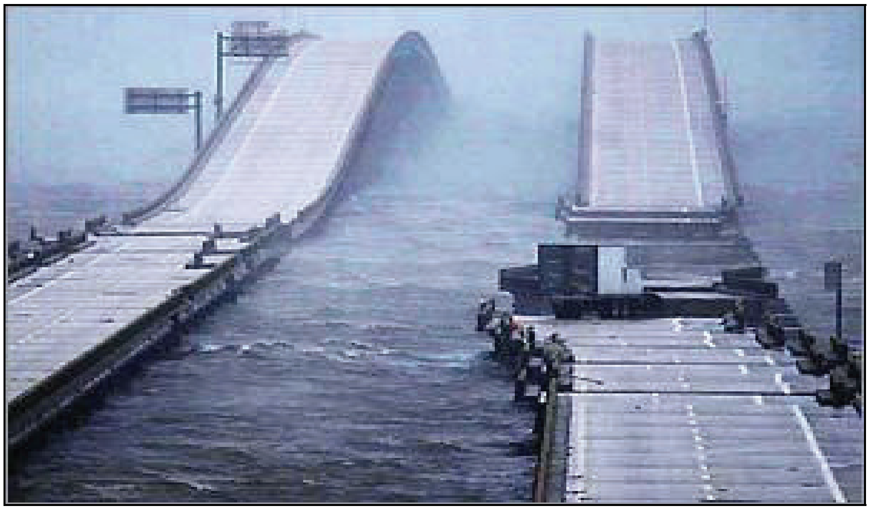
> ***"GRAND CHALLENGE 4: ADVANCING THE AASHTO SPECIFICATIONS***
> *To understand the limit states required for safe, serviceable and economical bridges and highway structures, and to develop enhanced reliability-based provisions addressing these limit states in a manner relatively consistent with traditional design practice and effort.*
>
> ***Anticipated Outcome:***
> *A stable and comprehensive LRFD Bridge Design Specifications that addresses all applicable limit states."*

The TRB Committee on General Structures recently authored the following statement as their vision for bridge design in the year 2020.

> *" To meet the challenges posed by the increasing demands placed on our structures infrastructure due to new technologies, globalization, and the unpredictability of natural and manmade events, the industry must continue to make advancements of new knowledge in such areas as:*
>
> *Consistent application of heavier and less predictable vehicular and environmental loadings as well as expected hazards in structures lifetime;*
> *Multi-hazard based designs and analysis for consistent safety and improved performance at least life-cycle-cost;*
> *Development of nondestructive methods for reliable and quantitative evaluation of infrastructure (such as pre-stressed and precast concrete structures and concrete bridge decks)*
> *Just-in-time maintenance and other operations based on smart structure and automated analysis philosophies;*
> *Enhanced ability to deal with economic considerations and future trends;*

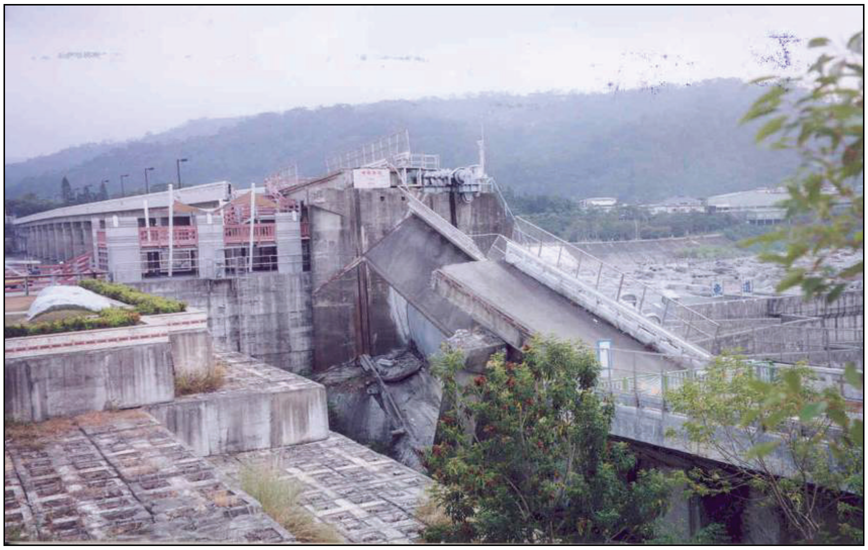
The Important Issues for us are to do our best to develop design specifications that will prevent the following:

- Storm Surge & Wave Action



- Flooding and Scour

- Earthquake

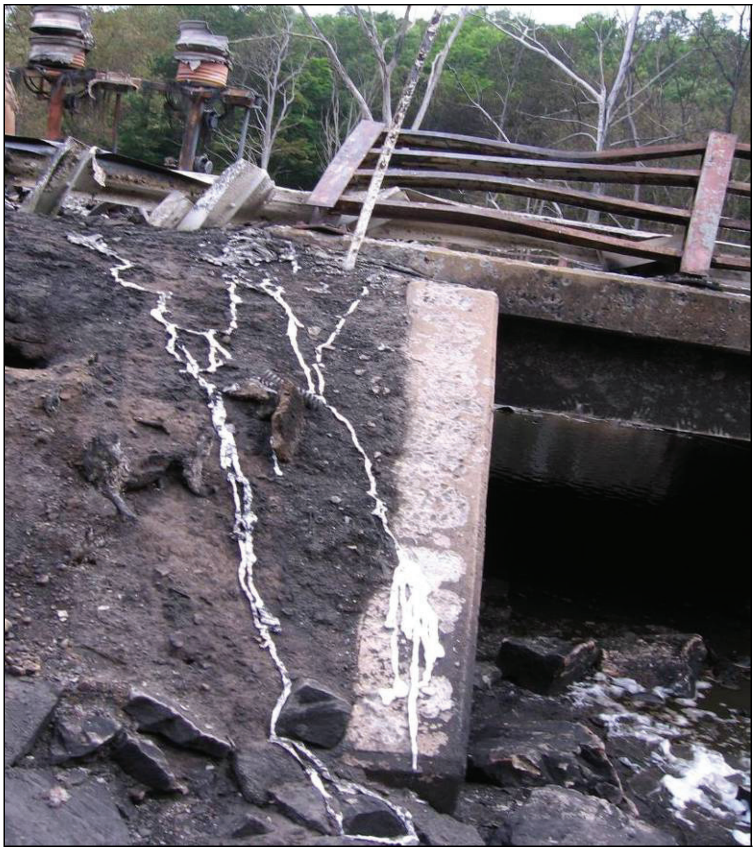


- Landslide/Washout

- Tsunami

- Blast Load

- Fire

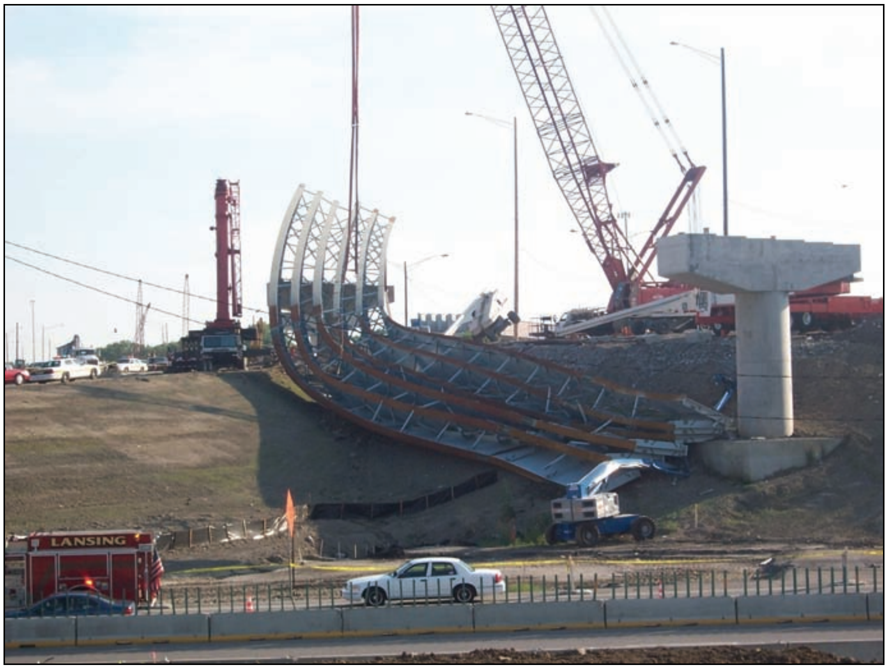


- Vehicle Collision

- Barge Impact

- Overload

- Construction/Erection Issues



The challenges are truly grand but the industry must continue to work to advance the current AASHTO specifications. It is very apparent that we must continue to strive to better represent loads that new designs and existing bridges will experience during their service life to best satisfy the public's expectations of a safe and efficient system of bridges on our country's highway system.

Hazard-Resistant Design from Code to Practice: The Viewpoint of a Bridge Designer

2

Thomas P. Murphy

The development of a Multi-Hazard approach to bridge design has yet to be well described in terms of impacts on the practice of bridge design. Therefore, rather than address details that do not yet exist, priorities and desires as to the future aspects of this approach can be expressed in the hopes that the resulting changes to design practice are for the most part positive, at least from a designer's perspective. Toward that end, the following issues will be briefly touched upon:

- Simplification of the design process
- Unification of design approach
- Relevancy to observed performance
- Enhanced understanding of risk

Today's designers are faced not only with increasing sophistication in the design specifications, but also greater demands that bridge designs respond to a growing list of additional constraints including:

- Aesthetics
- Durability
- Rapid construction/replacement
- Shortened project schedules
- Maintenance of traffic
- Tightening transportation budgets

Improving the performance of highway bridges to multiple hazards needs to be prioritized with the competing constraints noted above.

Simplification

The only load case that matters is the one that controls. Finding out exactly which combination of loads and factors constitute that case has, over the past half-century, become increasingly difficult. In the era before widespread availability of the PC, designers applied structural intuition, born of experience, to home-in on load combinations that would likely control the design. With modern tools available, and a much more comprehensive specification to work from, it is not unusual to have a design program or spreadsheet investigate upwards of 150 load combinations for a pier of even the simplest of bridges. The concept of Multi-Hazard design for bridges offers the opportunity to simplify that part of the design process, and identify early which hazard might control the design loads of the bridge. If the Multi-Hazard design approach can be structured in such a way as to make relatively quick comparisons early in the design process among the competing design hazards possible, this would at least not contribute to further complexity in the specifications, and perhaps even represent progress toward simplification. Hopefully what will not happen with a Multi-Hazard approach is an increase in either the number of hazards that need to be considered, or an increase in the number of ways those hazards must be combined.

Unification of Approach

It must also be kept in mind that writing a specification is only the beginning. Designers must implement the specifications, owners and regulators need to understand and enforce the provisions, and constructors need to build the structure according to the design intent. For the designers, it is often not enough just to be able to follow the equations in the specification, but they need to understand the behaviors the provisions are trying to ensure or avoid, as the case may be. As the hazards addressed become more complex, inevitably so must the designers' comprehension of the behavior evolve and mature. The ability to create a model of the structure and apply loads is not the same as the ability to understand behavior. This must be kept in mind when attempting to develop a comprehensive approach to bridge design hazards.

The Multi-Hazard approach to bridge design also offers the possibility of unifying the acceptable risk among all the hazards a bridge must be designed to resist. Typically, the level of safety against failure due to different hazards has been arrived at independently. As extreme load cases generally have a load factor of unity applied to them, the choice of the design load level, either through a probability of exceedence in a given length of time, or through a return period, determines the reliability of the design to that hazard.

These return periods for design hazards have been arrived at more or less by utilizing engineering judgment considering the cost of increasing resistance and the consequences of failure. Most striking are the differences in the return periods of the design events for flood, seismic, and wind events. The specification for the design of bridges subjected to coastal events, currently under development, uses a 100 year return period for determining the design wind speed and surge height to calculate loads on the structure. For seismic, the latest version of the seismic provisions for the AASHTO LRFD specification calls for designing to a 1,000 year return period level. In the past this has been as low as 500 years, and in the building sector the design event is 2,500 years. For typical wind design, a return period of 75 years is used. However, for long span bridges where stability issues arising from wind excitation are considered, return periods as long as 10,000 years have been specified.

Intuitively, there appears to be an inconsistency in treatment of differing hazards when return periods for the design level can vary by a factor of 100. The LRFD design philosophy is based on the idea that through the use of load and resistance factors, a uniform level of reliability can be achieved across different structural configurations and load types. For loading that occurs relatively frequently, such as live load, this is a straightforward task. The distribution of maximum live loads is known to a much higher level of certainty than is the distribution of any of the more extreme hazards such as earthquake or coastal storms. Even when the distribution of live loading changes due to fluctuations in the legal load limits, or truck design, the statistical data can be obtained by monitoring loads over a relatively short period of time. An issue that arises with extreme hazards is whether, given the relatively short amount of time over which reliable data has been collected, the true distribution of loads for long return periods can be known with any reasonable degree of certainty. One approach that has been applied in the past is to account for uncertainty when developing probabilistic loadings. This tends to increase loading intensities in areas where the least is known. A question can be asked as to whether this is the best approach, or whether the increased potential risk arising from the uncertain probabilistic data should be accepted without increasing the design loads.

Relevancy of Hazards

A review of historical failure data may be instructive in pointing to those hazards that may not have been adequately addressed in past design provisions. Figure 2-1, based on data compiled by the New York State DOT, shows the causes of bridge failures, as a percent of the total for a 39 year period. From the figure it is apparent that the single largest cause of bridge failures has been hydraulic. Collisions, both vessel and vehicular, account for the next largest percentage followed closely by overload. Earthquakes, the focus of much bridge related research and development over the last 20 or more years, are responsible for only 1% of failures.

In considering the figure above, it is important to note that the 39-year period is a very short time for hazards characterized by long return period events, such as earthquakes and also that the consequences of failures are not equal across all causes. For example, failures due to scour are likely to be isolated, affecting only one or two bridges per occurrence. Also, there are not likely to be concurrent failures in other sections of the regional infrastructure. For failure due to earthquakes, the situation is likely to be very different, with potentially widespread damage and secondary effects, like fire, for which the region's bridges need to function as part of a disaster response lifeline.

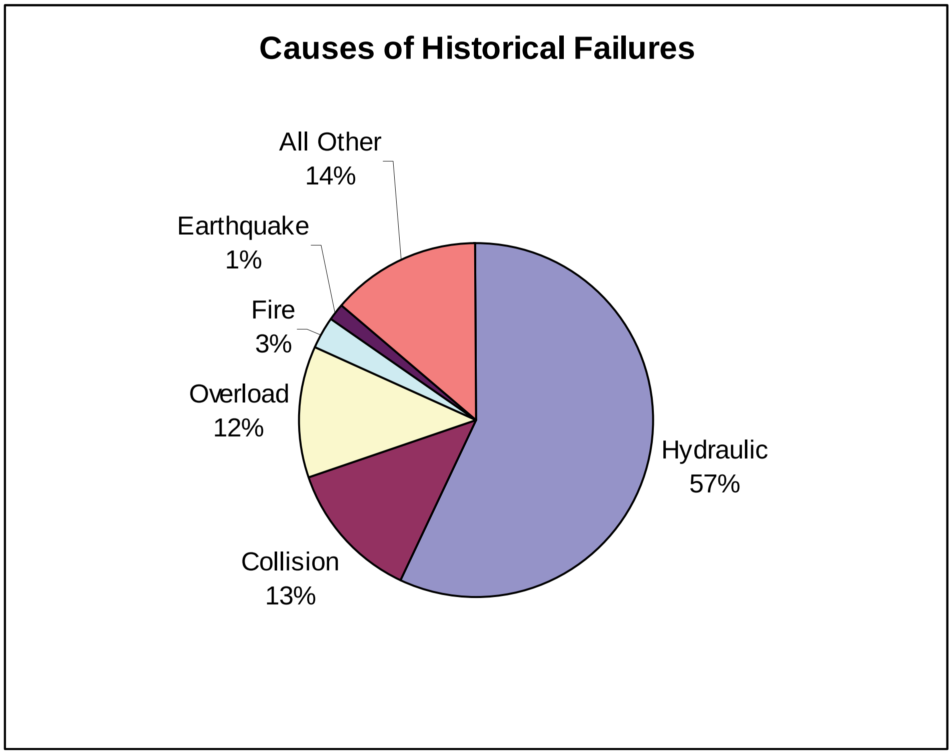


Figure 2-1. Historical Causes of Bridge Failures during a 39 Year Period

Enhanced Understanding of Risk

As the treatment of hazards in the specification grows more sophisticated, our understanding of the assessment of risk and the underlying probabilities must also grow. When speaking about the risk to a structure posed by a hazard, we often define a probability of exceedence during a given time period. This time period has usually been an approximation of the intended design life of the structure. For example, earthquakes are spoken of as having a 10% probability of exceedence in 50 years (equivalent to a 475 year return period). Using this logic, it would seem eminently reasonable to conclude that if a structure is intended to have a shorter life, a lower return period could be used while maintaining the same percent chance the design load is not exceeded during its life. This thinking can and is also applied to retrofitting existing structures, which may have only a few decades remaining in their useful life.

There is an inherent assumption in the above approach which is that the goal of the design process is to protect the structure against failure. An alternative view might be constructed based on the point-of-view of the user of the bridge. Normally, the two coincide. However, as the design life of the structure becomes shorter, there can be a divergence in the two evaluations of safety. A rather extreme case can be considered to highlight the differences. At a given bridge site, a "temporary" structure is constructed with a design life of only 2 years. Due to the short expected life, the hazard design loads (wind, flood, earthquake, collision, etc.) are much smaller than for a "permanent" bridge. Two years after the bridge is constructed, it is demolished and an identical one is constructed in its place. This is repeated for 50 or 75 years. From the bridge's perspective, this would be an acceptable, albeit very expensive, solution as the probability that any one of the "temporary" bridges would collapse in its lifetime is the same as the probability that a "permanent" bridge would collapse during its much longer lifetime. From the road user's perspective, however, the view is very different. Over any given period of time, the likelihood of an extreme hazard event occurring will be the same for both the "permanent" and "temporary" bridges. Should that event occur, the "temporary" bridge is much more likely to collapse, potentially resulting in the loss of life.

One question that can be asked in light of the above example is whether there should be some other measurement of safety that does not depend on the design life of the structure. This may become an important question in the development of the Multi-Hazard approach as it could have an impact on how differing hazard are compared, combined, and ultimately designed for.

Normal Function vs. Hazard Resilience in Bridge Design — View of a Stakeholder

3

George A. Christian

While multi hazard design theories are still to be developed, bridge owners and designers are still faced with assuring the safety for their bridges to such hazards, both in new designs as well as in maintaining their existing bridge inventories. Much of what has been done to date has been a result of a combination of experience, research, and engineering judgment. Design codes have evolved from considering only service limit states and elastic behavior to now addressing extreme event limit states and post-elastic or ultimate strength resistance. Lessons learned from actual failures have resulted in new design requirements being developed such as the seismic design provisions. Failures have also heightened the awareness of vulnerabilities that have led designers and bridge owners to incorporate failure resistant features into their structures, including load path redundancy, fatigue resistant details, scour resistant foundations and collision resistant substructures. The improvement in reliability that these features provide has not been quantified but they have become to be part of common practice.

New York State DOT Bridge Safety Assurance Program

As a follow up to the failure of the New York State Thruway bridge over Schoharie Creek in 1987 that was attributed to local scour of a shallow foundation during a flood event, New York State DOT initiated a Bridge Safety Assurance Program to bridge failure vulnerabilities that extend beyond normal deterioration and loss of structural function that are evaluated by traditional bridge inspection and load rating methods. The first step in this process was to identify such modes of failure for which vulnerability assessments could be developed. The failure considered based on surveys of actual bridge failures as well as from the conventional thinking at the time, and they included both vulnerabilities that could be attributed to prior design practices as well as what are generally considered as extreme events. The selected failure modes included: 1) Hydraulics and scour; 2) Collision, both vehicular and vessel; 3) Extreme overload; 4) Steel details; 5) Concrete details; and 6) Earthquake.

For each listed failure mode, a screening and assessment procedure has been developed to classify a bridge in terms of its susceptibility or vulnerability to failure caused by that mode. The vulnerability classifications are relative for that mode and there was no attempt at a calibration process to quantify the vulnerabilities among the different modes.

Individual failure vulnerabilities are compared in terms of a recommendation action to address the vulnerability. For example, what are thought to be the highest vulnerabilities were addressed by directed mitigation programs, notably scour retrofit programs on the most scour vulnerable bridges and steel detail retrofits or adding redundancy to the most fracture critical, fatigue prone steel bridges. Lesser vulnerabilities would not be prioritized for dedicated mitigation programs but are more likely addressed in the course of a normal capital rehabilitation project, including being considered as a justifying factor to replace a bridge vs. its rehabilitation.

While the New York Bridge Safety Assurance initiative was directed primarily at its existing bridge inventory, the concept and process is applied, implicitly as well as formally, to new designs. Essentially the commonly identified vulnerabilities are avoided in new designs. Typical new designs incorporate deep, scour resistant foundations; fatigue resistant details and load path redundancy were practical, ductile concrete details, seismic resistant design and details, and overload capacity.

Other Hazards to Consider

Since September 11, 2001, the threat of a terrorist attack on a bridge has been given much attention on a national level. Owner agencies have assessed the vulnerability of their bridges to such attacks, generally using procedures that assess the structure's vulnerability to various threats, the consequence of an attack and the importance or attractiveness of a bridge being a target. Most evaluations consider the primary threat to be explosive devices, leading to active research and applications on blast loads, resistance, and mitigation. Some of this work has attempted to build on seismic resistance concepts to address blast.

Fire is another hazard to bridges that has gained attention due to a number of incidents nationwide where bridges were damaged or destroyed by fires from accidents involving gasoline tanker trucks. While considered an extreme event, fire affects the bridge's resistance or integrity as opposed to being an extreme loading. Fire is still considered a rare, low risk event that is unlikely to change the design standards or criteria for bridges. Attention will be given to designing and improving bridge and roadway geometries and railing/barrier systems as well as relying on operational measures (e.g., speed enforcement, vehicle safety inspections) to reduce the potential for such accidents.

Other extreme events that are applied loads are due to ice, wind, and wave action. Wind and ice loads are normally considered in current designs where conditions apply and present design specifications address these loads. Wave action has received new attention for bridges as a result of the damage to bridges during Hurricane Katrina and the recent hurricanes in Florida.

Key Issues for Multi-Hazard Design

Presently new bridge designs, as well as the assessments of existing bridges, address each failure hazard individually. There has been no significant attempt to consider or classify the effects of multiple hazards in the normal bridge design or evaluation process. This is due to the difficulty of the problem, the perceived low level of risk compared to the design implications of hazards taken individually, or level of risk relative to the effects of normal design loads along with durability issues. Much of today's extreme hazard mitigation is done somewhat intuitively through the use of good detailing practices more so than analytical design concepts.

Some work has been done to compare the effects of multiple hazards on structural response for buildings. Figure 3-1 shows schematically how different hazards fall into different ranges in terms of response as measured in terms of amplitude and frequency (Ettouney et al., 2005). These measures indicate little overlap, suggesting challenges in the analysis and in developing design strategies to effectively address such multiple hazards.

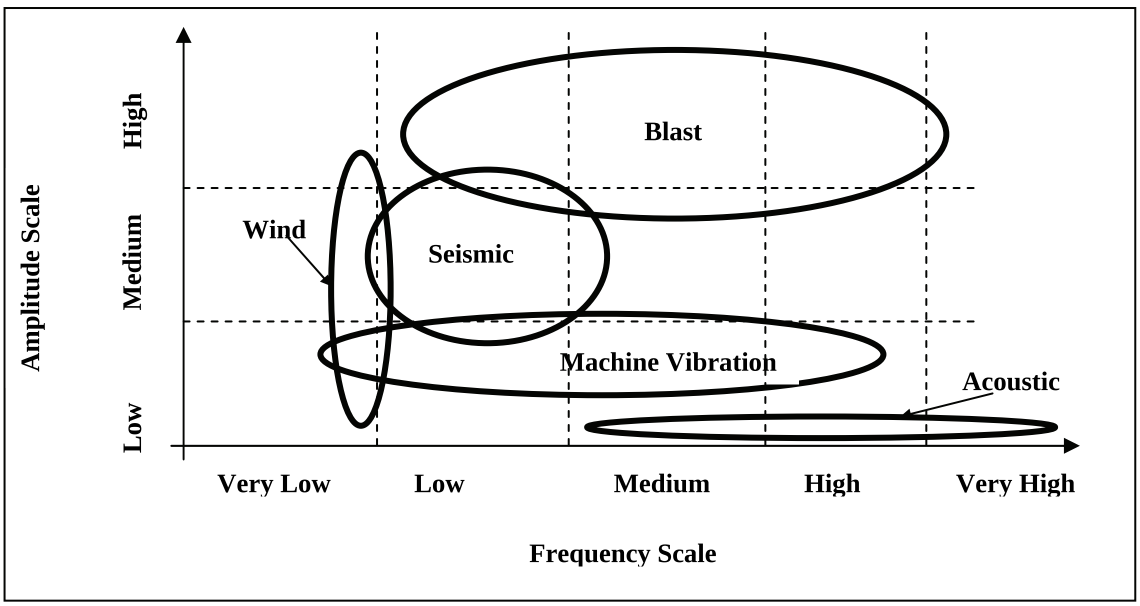


Figure 3-1. Qualitative Frequency-Amplitude Distribution for Different Hazards

The recent interest in developing designs to resist both seismic and blast effects has identified some of these challenges. In simplest terms, structural mass (weight) creates greater internal seismic forces, while structure mass reduces the internal forces from a blast load. The challenge is to optimize a design to account for the conflicting hazard effects. At the same time, this optimization needs to be more than a compromise that is not effective for either hazard.

The designer would also consider the relative risk of each hazard, which may skew the design to be more resistant to one hazard vs. the other. Research is needed to better quantify the relative risk among extreme event hazards so such comparison can be more intelligently made. The large difference in return periods used for individual design loadings, e.g., 1000-year earthquake, 100-year storm, and 75-year vehicle load suggest the need to better calibrate the risks of failure from each. Calibration that provides consistency in terms of relative risk would be one development that would potentially provide for more efficient designs.

Most bridges are typically designed for primary load cases, e.g., dead loads plus live loads for bridge superstructures, and then checked for the less frequently occurring load cases, including extreme event loads. For substructures and foundations, the typical design load case is a combination of gravity and lateral loads due to wind and horizontal forces transferred from the superstructure. The newer seismic design provisions have resulted in seismic loads being treated as a primary "design" load case that is expected to control most substructure and foundation designs, notably in higher seismic areas where the seismic risk is accepted to be relatively high. If multi-hazard design criteria result in more such loads controlling typical designs, there would be a cost involved that similarly must be weighed relative to the risk.

The effects that multi-hazard designs will have on normal bridge performance must also be considered. In most cases, improving the resistance to hazard effects should result in some form of hardening or added capacity, which may likely benefit performance and durability under normal loads and conditions. As noted, some design features may not be complementary in resisting different hazards, e.g. seismic vs. blast or collision. It is also possible that such design features could be detrimental to performance under normal conditions. For example, to economize on the increased foundation sizes due to seismic loads on solid (collision resistant) bridge piers, some designers relocated fixed bearings from the piers to one of the abutments. A negative aspect of this configuration is necessity of providing all the expansion length at the opposite abutment, necessitating larger bearing and deck joint movement capacities. Solutions to counteract other hazards may include tie downs, restraints or dampers that may adversely affect normal bearing performance under thermal and live load deflection effects, and add to normal maintenance needs. Another solution for situations similar to that described above has been to use integral abutments with expansion bearings at all the piers. Integral abutments are very effective in terms of construction cost and normal maintenance. Initial installations have shown to have good performance under normal loads but they have yet to be subjected to extreme hazards. Research to predict and validate the effectiveness of integral abutments subjected to multi-hazards would be useful.

While research is necessary to establish realistic, risk based multi-hazard design, these hazards will be considered as extreme and their mitigation should not be detrimental to normal bridge performance and maintainability. The cost impacts of such designs must be weighed against the risk and there should be some level of consistency of risk. Hazard mitigation should also consider the application and performance of cost effective resistant details, in addition to establishing analytical loads. Establishing the effectiveness of such details against multi-hazards should be a primary objective.

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A Probability-Based Methodology for Comparison of Extreme Hazard Loadings for Bridge Design

4

Mai Tong and George C. Lee

Occurrences of extreme hazard events in recent years have raised serious concerns on the vulnerability of critical buildings and infrastructure systems under natural and man-made hazards. Highway bridges – one of the critical elements of surface transportation infrastructure of society - are among those at the highest risk.

Despite increasing interest in strengthening highway bridges against extreme hazards, substantially modifying or increasing current design provisions is a complex process that requires the consensus approval of the entire design community. Before starting this process, at least one area that needs a better understanding is that of extreme hazard loading. In order to revise overall criteria to select the proper design hazard loadings, it is necessary to have a methodology for systemic comparison and assessment of all potential hazards and their impacts. To do so, a variety of issues must be evaluated, such as

- Probability of hazard occurrence
- Hazard intensities
- Hazard loading effects on bridge components
- Possible damage states and failure modes
- Consequences of a hazard event to functionality, serviceability, repair cost, and other indirect cost and long term impact

Due to the lack of accurate information and the complicated dependency of these issues on other factors, e.g., available engineering solutions, public perceived risk, cost of mitigation, competitive priorities (Hwang et al., 1988; Wen and Kang, 2001), there is not currently a generally agreed upon methodology for comparison and assessment of hazards. Neither is there a clearly identified set of key parameters for evaluating multiple hazard loadings for highway bridges. This dilemma directly restricts the bridge engineering design community in addressing the following key question: "What hazards should be more emphasized in highway bridge design and why?"

In the AASHTO LRFD Specifications (AASHTO, 2004), the design limit states are classified in four categories:

- Service limit states

- Fatigue limit states
- Strength limit states
- Extreme event limit states

Since structural safety is classified into strength and extreme event limit states, the design criteria tend to control the failure probability of these limit states to a minimum within practical constraints. The extreme events known to cause severe structural safety concerns of highway bridges include earthquake, severe wind, vessel and vehicle collision, scour, flood and wave surge, traffic overload, fire and explosion. Design requirements against these hazards have been developed and are included in the AASHTO Specifications. Yet, whenever bridge damage or collapse occurs in an extreme hazard event, one of the questions of investigation is whether these bridges had been designed with sufficient hazard resistance or whether there are new problems beyond the original design code consideration. In many cases, the answer can largely vary depending on how the risks associated to the hazards are perceived.

For example, frequency of occurrence of these hazards can be briefly probed from two statistics: declared disasters and hazard induced bridge collapse. According to FEMA's NY State disaster declaration record from 1965-2006 (*http://www.fema.gov/femaNews/disasterSearch.do*), there were a total of 51 State declared disasters during this period. Among these extreme hazards, there were 39 flood events, 8 hurricanes, 1 earthquake, 7 snowstorms, 2 terrorist attacks and 1 landslide. Although not every natural disaster is a serious concern of bridge safety, these statistics show that the design considered hazards are consistent with the occurrence of potential disasters that can cause a safety threat to highway bridges. This statistics also indicates that flood events are the most frequent hazard followed by hurricane, tornado and snowstorm in New York State. These statistics vary from State to State in terms of recurrence rates and severities.

Based on separate statistics from NY State Department of Transportation, nationwide bridge collapse events from 1969~ 2006 are tabulated against different causes in Table 4-1. The table shows that flood is the number one cause of bridge collapse with 718 events, followed by traffic overload with 220 events, vessel collision with 36 events, earthquake with 19 events and wind with 8 events. This data suggests that while flood is still the leading devastating hazard, vessel collision and earthquake have a higher potential to bring collapse to highway bridges than wind.

Table 4-1. Hazard Induced Bridge Collapse from 1969~2006

	Flood	**Traffic Overload**	**Vessel Collision**	**Earthquake**	**Wind**
Number of bridge collapses	718	220	36	19	8

Source: NY DOT

Uncertainty of Extreme Hazards

The above statistics provide evidence that extreme hazards cause significant damage to highway bridges. However, in order to quantify the risk for design purposes, deterministic mathematical models cannot handle the inevitable uncertainties. Various stochastic models have been employed to represent hazards and particularly extreme hazards by other researchers (Ghosn et al., 2003) for their maximum possible loadings. A well-received method of using probabilistic models in engineering design is the reliability theory based LRFD methodology, which translates the structural design safety requirement as a uniform reliability requirement, which can be directly measured by a structural failure probability.

If the LRFD methodology is considered as the desirable approach for determining the level of design resistance against extreme hazards, one difficulty a bridge designer faces is the large uncertainty associated with hazard loadings. Due to the lack of supporting data, calibration of hazard loadings inevitably carries large uncertainty regardless of the model used.

The following table compares the COV of earthquake, wind, scour and vessel collision loadings with the live load in normalized mean values.

Table 4-2. Comparison of Covariance of Different Loading Source

	Live load (S_{75}/S_{HS})	**Earthquake (F_{EQ}/F_{EQn})**	**Wind (W/W_n)**	**Scour (SC/SC_n)**	**Vessel Collision (CF/CF_n)**
Bias	1.85	0.6	0.78	0.37	0.1
COV	15%	138%	37%	49%	176%

In Table 4-2, X_n represents the nominal design load. The ratios are calculated based on the actual loads versus the nominal load. It is seen that except for the live load (where S_{75} - maximum truck load of 75 years – is 1.85 of its nominal live load, which suggests that the hazard load has covered a sufficiently wide range of the possible actual load conditions, for all other hazard loads), the actual load values have a bias that is less than 100% of their nominal load, which suggests that the nominal model value carries a large margin of uncertainty. In addition, the corresponding COVs of hazard loadings are much higher than for live load.

In view of the above statistics and the uncertainty comparison associated with extreme hazards, the following general understanding about extreme hazards and their potential threats to structural safety of highway bridges can be derived:

1. Hazard loadings from extreme events remain a major safety concern in bridge design
2. Based on the recurrence ranking of the most commonly known extreme hazards, flood, wind and earthquake are the three top extreme natural hazards

3. Comparison of different extreme hazards needs a comprehensive platform.
4. There is a large uncertainty associated with the modeling and prediction of extreme hazards for both intensity and occurrence.

Return Period and Extreme Hazard Comparison

In the AASHTO LRFD Specifications, the design extreme hazards are treated as comparable to their return periods. For example, a design earthquake is set to 475- or 2500-year return periods for regular and important bridges, respectively; design windstorm speed is set to a 50-year return period or 100 mph for areas that lack sufficient data; scour is set to a 100-year return period and design vessel collision is set to annual probability of collapse less than 0.001 or 0.0001 for regular or important bridges, respectively, which correspond to a 100- or 1000-year return period. As a comparison, the design live load is set to the maximum of a 75-year return period.

Since return period is a straightforward concept for hazard comparison, a natural question is raised: Can all design hazards be calibrated to the same return period such as the 75-year design life span of the bridge? Such an approach, if feasible, has an advantage in that it provides a uniform measurement of the recurrence rate for all hazards, and is consistent with the philosophy of LRFD methodology to provide a uniform probability measurement of the structural reliability.

The answer to this question is not so straightforward. First, the onset of a hazard to damage bridge structures is influenced by many factors, and there is not sufficient knowledge to predict many of them. Some commonly considered factors in bridge design include

- Frequency of hazard occurrence
- Hazard size, intensity, area of impact
- Local site conditions
- Combined hazard events
- Uncertainty of hazard measurement

In addition, hazard induced structural damage or failure depends on the performance of critical structural components, design details and their vulnerabilities under different hazard loading effects. Typically, they may include:

- Strength of critical components under hazard loadings
- Consequences of bridge damage by different hazards
- Redundancy and ductility
- Risk tolerance implied in selected design loads
- Construction quality
- Materials' engineering properties

It is evident that return period can only provide the information on frequency and recurrence, and is an oversimplified measure of extreme hazard events.

A Comprehensive Comparison of Multiple Hazards for Bridge Design

If return period is an oversimplified comparison reference, is there a better way to compare different extreme hazards without overcomplicating the problem? From the above discussion, it seems that a comprehensive comparison of hazards based on a few added key parameters may suffice. In short, in addition to the hazard return periods, there are four other pieces of hazard information that are generally useful for structural engineering design.

The first is the magnitude and footprint of a hazard event. Many natural disasters such as earthquake, hurricane, and flood cover a large area. Without reviewing the magnitude and footprint, the various engineering decisions for selecting different levels of design resistance for various hazards cannot be explained. Therefore, the maximum possible magnitude of a hazard in a region is necessary information for macro risk assessment.

The second is the local intensity of a hazard. After all, direct hazard loadings to bridge structures are largely related to the local geophysical and environmental conditions in addition to the potential hazard sources. Structural design also requires the maximum possible intensity of a hazard at a specific site. Because of the large number of influential factors and the lack of sufficient historical data of extreme hazards, estimation of local hazard intensity involves large uncertainty. Proper quantification of such uncertainty is significantly helpful for engineering design decisions.

The third is the comparison of the local maximum possible intensity of a hazard with the maximum considered intensity for an individual bridge. If this information is available for a group of hazards, it helps to compare and identify, from the viewpoint of bridge design, which hazard is still short of sufficient protection. Naturally, dominant hazards are the ones such that their potential maximum intensities are higher than the design considered intensities.

The fourth is the level of risk tolerance implied by the design standards. Extreme hazards are random events and in general, there is always a chance that a hazard of extremely high magnitude may occur or two or more hazards may occur at the same time or in sequence; therefore, the bottom line question is whether it is worthwhile to design a bridge for a severe hazard or combination of hazards with very small probability of occurrence. As an alternative, taking the risk and carrying out proper preparedness and response planning is another viable and practical approach. The current AASHTO LRFD Specifications intend to calibrate the load and resistant factors for the safety limit states to correspond to a reliability index $\beta = 3\sim3.5$ (Nowak, 1999). This implies that a risk tolerance for extreme hazards is controlled under 0.03%.

A Uniform Hazard Comparison Procedure

To pursue the above described design information, a probability-based comparison and evaluation procedure for hazard loadings is proposed. The main focus of this procedure is to integrate the aforementioned comprehensive comparison of hazard into a uniform platform. The platform is an extension of the return period reference and is compatible with the LRFD methodology.

The proposed procedure consists of four steps:

1. Determine the maximum hazard magnitude and footprint for the region of interest.

   Magnitude of a hazard is a quantity that is associated with the probability of occurrence (or return period). Therefore, maximum magnitude cannot be determined with 100% certainty. It usually corresponds to a probability of exceedance or return period, for which the magnitude versus return period function is typically obtained by a statistical curve-fitting of historical hazard data and a mathematical extrapolation of the function to a wide time range beyond the historical data originally covered. Such a mathematical model can produce a nominal magnitude for any given return period. However, it often lacks sufficient physical evidence to support the model predicted nominal magnitude for the extended return period. Perhaps a better way to improve such a mathematical model is to introduce state-of-art knowledge to identify a physically possible maximum magnitude based on current hazard research advances. For instance, the maximum magnitude of earthquakes in California is currently estimated to be 8.0~8.5 on a moment magnitude scale according to Field et al. (2008), which incorporated both a statistical model and a seismological characteristic model.

   The regional maximum magnitude and footprint of a hazard has two immediate uses in this procedure. It provides an upper bound constraint for the maximum site intensities within the region as described in Step 2 (below) and is an important reference for determining the risk tolerance level as described in Step 4 (below).

2. Determine the local maximum hazard intensity and associated uncertainty rating

   As previously mentioned, magnitude is useful for regional or area zoning and planning, but individual bridge design requires the site specific hazard intensity information. Also, certain hazardous events such as vessel collision and traffic overloading may be isolated events that do not have area or regional impact. In general, either the magnitude versus return period relationship can be modified to give intensity versus return period relationships, or some self-contained hazard intensity relationships are available. However, from the maximum magnitude to maximum local intensity, the influence of many local factors adds more uncertainty. This is particularly significant for extreme hazards as they all correspond to the so called tail events, which are the stochastic events with very small probability of occurrence (the probability is the integration of a right-end tail of the probability density function). Therefore, it is necessary to introduce a confidence level to measure the potential volatility of the model-predicted maximum possible intensity (MPI). For different application purposes, this confidence level may be set at 50%, 90% and 95%.

Note that magnitude and intensity information is typically provided by hazard research organizations such as USGS and NOAA. Engineers and practitioners should be aware of the latest information for the hazard loading evaluation and comparison, but do not need to generate this information themselves.

3. For a specific failure mode that a hazard may cause, compare the ultimate strength of the bridge with the load demand by the local maximum hazard intensity

   From a structural designer's point of view, hazard resistant design is to prevent critical failure modes from occurring when the structure is subjected to extreme hazard loadings. Design of a structure's strength is determined in the process of searching for an optimal structural system to satisfy the various design limit states. In terms of the LRFD methodology, the design objective is implied in the target reliability index β or the control of failure probability P_f (Kulicki, 2005). This is directly related to the proposed uniform procedure explained in the following paragraphs.

   If a particular failure mode of a typical highway bridge is identified to be possibly caused by several hazard events (e.g., a shear failure of a bridge pier under earthquake, severe wind, and vessel collision), according to fragility theory, for each corresponding hazard there exists a damage state curve. This damage state curve represents the relationship between severity of damage and the corresponding hazard intensities. There are several different classifications of damage severity. Table 4-3 shows bridge damage states defined in HAZUS-MH (FEMA, 2005). Translation of damage states to performances (e.g., repair cost, down time) for seismic bridge design have been studied by Mander and Basöz (Mander and Basöz, 1999).

   Damage state is usually associated with a probability of certainty, which describes the probability that an actual damage exceeds the given damage state under a same level of hazard intensity, also referred to as risk probability. The acceptable level of this risk probability is to be determined by consensus of the engineering community. For example, a 10% risk probability of collapse for maximum considered earthquake (MCE) has been considered by seismic building design (Deierlein et al., 2007). Different levels of certainty will correspond to different damage state curves. For a group of hazards, if their damage state curves are calibrated at the same level of certainty, they can be compared with respect to a selected level of damage severity. This comparison is conceptually shown in Figure 4-1. The intersections of the damage state curves with the chosen damage severity level give the corresponding hazard intensities for which the bridge is capable to resist up to the damage state. If the comparison is based on the collapse state (DS5), the above determined hazard intensities are referred to as Collapse Hazard Intensity (CHI) for each hazard. Note that CHI can be chosen under different confidence levels. Some more detailed classifications of damage states are also provided in Chapter 5. The comparison cannot be directly carried out among different hazard intensities unless other commensurable grounds are established. However, each CHI can be compared against its maximum possible intensity (MPI) identified in Step 2 (above). Finally, there are three possible outcomes from each comparison:

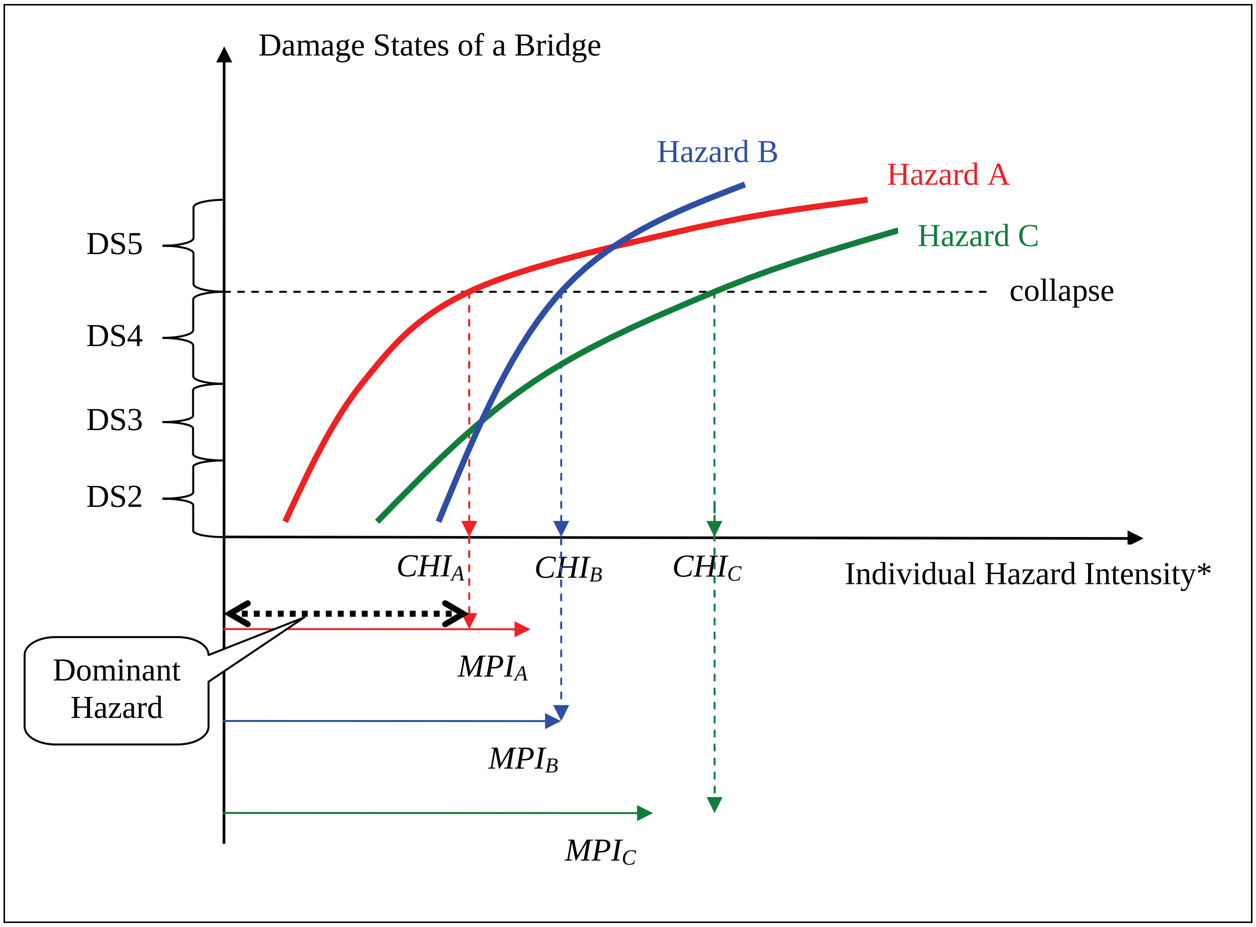
1. MPI is greater than CHI: This outcome implies that for this hazard, the bridge may still need more design protection to resist the CHI. The difference between MPI and CHI is the additional uncovered hazard exposure.
2. MPI is less than CHI: This outcome implies that for this hazard, the bridge is over-protected with respect to the CHI.
3. MPI is the same as CHI: This implies that for this hazard, the resistance is optimal with respect to the CHI.

Table 4-3. Bridge Damage States (FEMA, 2005)

Damage State	Descriptions
Collapse (DS5)	Any column collapsing and connection losing all bearing support, which may lead to imminent deck collapse, tilting of substructure due to foundation failure.
Extensive damage (DS4)	Any column degrading without collapse – shear failure - (column structurally unsafe), significant residual movement at connections, or major settlement approach, vertical offset of the abutment, differential settlement at connections, shear key failure at abutments.
Moderate Damage (DS3)	Any column experiencing moderate (shear cracks) cracking and spalling (column structurally still sound), moderate movement of the abutment (<2"), extensive cracking and spalling of shear keys, any connection having cracked shear keys or bent bolts, keeper bar failure without unseating, rocker bearing failure or moderate settlement of the approach.
Slight Damage (DS2)	Minor cracking and spalling to the abutment, cracks in shear keys at abutments, minor spalling and cracks at hinges, minor spalling at the column (damage requires no more than cosmetic repair) or minor cracking to the deck.
No Damage (DS1)	

Note that outcome 1 (above) often implies that the hazard is a possible dominant hazard among others since it is potentially a primary concern of bridge failure. However, to determine the dominant hazard, in addition to evaluating potential hazard damage and intensity, its probability of occurrence must be considered as explained in Step 4.

The design hazard intensities (DHI) currently set by the codes are typically lower than the CHI. However, these design check points are important to estimate the structure's performance at the ultimate state. Control of design points with respect to the CHI may offer an alternative system design criterion (Lee et al., 2008). In this regard, a ratio of DHI over CHI is a useful index for assessment of various expected performances of the structure under the design hazard intensities. When this index is normalized with respect to its inherent uncertainties, it provides design performance information for different hazards. This topic is to be addressed by the authors in a separate paper.



* Note that measurements of hazard intensities are different by hazard

Figure 4-1. Procedure to Identify the Dominant Hazard for a Bridge Structure

One of the challenges in setting targeted system performance to prevent structural collapse is that the linkages between various component failure modes and the structural system performance are weak. While a structural system collapse can always be traced to a critical component failure, it is not the case that every component failure will result in a system collapse. Efforts in performance-based design research have been focused on quantifying the structural system behaviors under CHI and also determining CHI in more accurate terms. In this regard, displacement-based design has recently advanced this effort to connect the structural collapse state to the capacity of lateral displacement. While this approach has some success in linking certain component failure modes to the system collapse state, it has been recognized that bridge collapse, which is a state of loss of load bearing capacity, is not always due to the loss of capability to lateral displacement.

4. Determine the level of risk tolerance

   Extreme hazards include both rare individual hazard events and two or more hazards occurring simultaneously or in consequence. In some cases, one hazard event may trigger other hazards, which are referred to as multi-hazard events. From a practical point of view, when considering such events, it is not justifiable to apply engineering mitigation for all those extreme hazards with too small a probability of occurrence; preparedness and contingency plans for emergency responses are often better alternatives. Therefore, before conducting hazard loading evaluation for structural design, a proper level of risk tolerance should be examined to ensure that risk mitigation efforts satisfy the cost-benefit requirements.

   Risk tolerance is often determined based on the hazard magnitude and footprint and possible consequence or secondary hazard events and the importance category of the bridge in the area. For regular bridges, a level of risk tolerance can be simply set based on an acceptable failure probability of the bridge structure. Corresponding maximum hazard intensities can be subsequently determined according to the chosen risk tolerance. For important or signature bridges, the acceptable risk tolerance must consider the regional impact of the hazard and functional loss of the transportation network if the bridge is down. Therefore, the risk tolerance is not a uniformly set value for all bridges.

   A dominant hazard for a bridge is the one that bears a potential risk above the risk tolerance level and has a MPI larger than CHI. Dominant hazard may vary from bridge to bridge even when hazard conditions are similar, since it is based on both hazard potential and vulnerability.

   One of the technical issues related to the selection of the proper risk tolerance level is the tail sensitivity problem. As occurrence of extreme hazard events is typically represented by the upper tail of hazard model (Probability Density Function PDF) and the uncertainty of the tail portion of PDF can be magnitude higher than for the statistical parameters such as mean and standard deviation, the high sensitivity of calculated total risk with respect to the uncertainty of the tail of PDF can undermine the validity of the risk-based criteria. Put it in simple terms, when considering an event with a small probability of occurrence, large uncertainties in the modeling and calibration data may result in false conclusions such that the assumed risks may subject to too large variations that prevent sound and reliable criteria to be derived. A study of this problem and its associated meaningful implications is presented in Tong et al 2008. .

Concluding Remarks

In this chapter, the challenges of extreme hazard comparison and assessment are reviewed. As an extension of the return period concept for extreme hazard comparison, the need to develop a probability-based procedure for hazard comparison and evaluation is articulated. This procedure is focused on identifying the dominant hazard for a bridge through a comparison of a group of different hazards. Such a procedure, once established, will be useful in several applications, including the improvement of the LRFD specification for highway bridge design against extreme hazard loadings.

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The Bridge Owner's Dilemma with Less Common Events

5

Susan E. Hida

Abstract
Bridge owners must choose the level of earthquake loading to design for, and whether or not to combine seismic force effects with those from coastal storms or other less common events. A cost savings may be achieved with lower design criteria, but the owner needs to know the level of risk and anticipated performance levels associated with such bridge engineering decisions. This paper outlines a methodology for hazard calculation appropriate for high-level decision-making on seismic performance and whether there is a need to combine seismic and storm loading. The method could potentially be applied to other infrequently occurring events.

Introduction

Funding for a new or seismic retrofit bridge project is influenced by needs for other public works such as energy generation facilities, jails, schools, wastewater treatment plants, to name a few. Setting priorities can be subjective and is further influenced by changes in revenue sources and political administrations. When a commitment is made to build a bridge, the resources available should immediately be compared to the owner's vision of the bridge

- Upon completion, i.e. architecturally
- In years to come with respect to maintenance costs
- During natural or manmade catastrophic events

This paper focuses on the latter and is extendable to structural retrofit for such 'extreme events.'

The owner must not only decide to design a bridge to withstand seismic or hurricane or blast forces, but also decide what level of damage could be acceptable. If compromises in bridge performance and operational objectives can be made, designing for an extreme event can be fiscally viable. This paper assumes that the owner will use non-collapse design criteria for whatever extreme event is being designed for. In other words, damage is acceptable so long as collapse i.e. loss-of-life is prevented. It is important to note that failure of one or more members is acceptable so long as the SYSTEM survives. Therefore, the calibration used for the *AASHTO LRFD Bridge Design*

Specifications (2007) is no longer applicable because the target reliability, β_T = 3.5, is based on the strength of a SINGLE girder (Nowak, 1999).

The comparative risk for various degrees of collapse given various qualities of bridges subjected to various seismic events, is developed herein. By studying case histories of damage in a given structure with a given level of vulnerability incurred during a given-level earthquake, probability-based hazard tables can be created. An engineer can then easily communicate the risk and ramifications of choosing a given level earthquake for design of a structure, to the bridge owner.

Finally, this paper uses simple, classic methods to evaluate conditional and combined probabilities. The occurrence of earthquakes is modeled using a Poisson distribution, and storms using a binomial distribution. The design life of the bridge is taken as 75 yrs, as used in establishing the HL93 vehicular live load. A hazard level is determined directly, circumventing the need for a reliability index.

Background

After the 1987 Whittier and 1989 Loma Prieta earthquakes, it became obvious to officials at the California Department of Transportation (Caltrans) that design or retrofit of all 12,000 bridges to withstand little or no damage during a seismic event would be cost prohibitive. To be good stewards of public resources, the need to accept the inconvenience of post-earthquake repair work—should such an event occur, was obvious. Only toll bridges and a select list of other structures on emergency routes were selected to have serviceability as its performance objective. For the majority of structures, geometric or material nonlinearity became acceptable and the goal became "no loss of life." Good detailing practices could confine damage to acceptable locations and prevent the structure from collapse: confining longitudinal reinforcement in concrete columns and piers, strategically locating plastic hinges for energy dissipation, and having a structure geometrically balanced for ideal behavior during an earthquake or other infrequent event (Caltrans, 2006).

Successful design or retrofit of a bridge requires that the sum of the factored loads be less than the factored resistance: $\Sigma\gamma_i Q_i \leq \phi R_n$ (AASHTO, 2007). For the calculations to be reliability-based, the resistance-side of the equation must be balanced with the loads-side in order to achieve an acceptable margin of safety. This balancing becomes more complex when non-collapse criteria are used for extreme events. The resistance of a SYSTEM must be quantified (right side of equation) and compared to the total load (left side of equation). A multi-column bent, for example, can be designed for a displacement of so many inches or kips of force. However, earthquake forces can occur in different directions. In a multi-span bridge, any bent may reach its collapse point first i.e. there are many potential collapse scenarios. Similarly, a "scoured-out" substructure can become unstable at different locations. A smaller load or displacement can be more critical for a potential failure configuration than a larger load or displacement in a different location. The science of applied SYSTEM reliability has not yet advanced to be of assistance to bridge owners, and generalizing the nonlinear structural resistance isn't realistic in the opinion on the author.

During development of the AASHTO LRFD load combinations, the authors of the Specifications merely set aside those pertaining to infrequent occurrences and called them "extreme events." There is no consistency, correlation, or connection between seismic (hazard maps based on 1000-yr event), scour (100-yr design flood; 500-yr check flood), and wind (designer may choose their own recurrence interval in establishing wind velocity at 30 ft above ground or water) in part because the public has a different perception of each. Furthermore, one has little warning to get off of a bridge during an earthquake compared to a storm. Caltrans uses approximately a 500-yr seismic event for bridge design, but feels that a 500-yr check flood is too severe. A 100-yr event is used with total scour and the fully factored Strength I Load Combination; the bottom-of-pile cap may be exposed so long as the piles are designed accordingly (Caltrans, v03.06.01). (AASHTO calls for the Strength I load combination to be used with load factors of 1.0 and scour due to a 500-yr event.)

Modeling of Extreme Events

Earthquake

Seismic events are modeled using an exponential distribution in the time domain because the following assumptions related to a Poisson process are satisfied (Ang, 1975):

1. Events occur at random over the life of the bridge,
2. Occurrences of events occurring in non-overlapping intervals are statistically independent (aftershocks to major seismic events, as well as storm surges due to major seismic events i.e. tsunami, have not been considered),
3. The probability of occurrence in a small interval is proportional to the interval, and the probability of multiple occurrences in a small interval is negligible.

Let $X_{[0,t]}$, be a random variable denoting the number of rare events in a certain time period, $[0,T]$, and be equal to the value x ($x = 0,1,2,...$). If T_1 is the waiting time for the next event to occur:

$$P(T_1 \le t) = 1 - e^{-\upsilon t} \tag{1}$$

where ν is the rate of occurrence, or reciprocal of the return period and t represents the design life of the bridge. Probability values for various return periods are shown in Table 5-1.

As with most earthquake quantifiers, return period of a seismic event originating from a given fault is a function of that fault's slip rate. Because slip rate measurements can differ at different points along the fault, considerable debate exists as to whether or not return period accurately portrays a seismic event's potential force. Slip rate measurements are improving as GPS measurements become available, more data points are accumulated, and investigation is done deep enough to incorporate historical events. The calculations herein, then, are as accurate as the quality and breadth of data used to determine slip rate.

Table 5-1. Probability of Occurrence During 75-yr Bridge Design Life for Various Events

Return period (RT in years)	**Probability**
50	0.78
75	0.63
100	0.53
150	0.39
300	0.22
500	0.14
1000	0.07
2000	0.04

The possibility of various-sized events occurring during the time t need not be considered because the design life of a structure is small in relation to time between events of catastrophic magnitude. Furthermore, the author's objective is to arrive at one design event appropriate for the level of risk the bridge owner is willing to assume. To use a rate of occurrence of all pertinent events isn't appropriate because a bridge isn't designed for simultaneous earthquakes or cumulative seismic damage.

Storms

Storms fit the characteristics of a Bernoulli sequence and are modeled using a binomial distribution:

1. Two possible outcomes—occurrence or nonoccurrence
2. The probability of occurrence in each trial is constant
3. The trials are statistically independent

If 100- and 500-year recurrences are used, the annual probability p is 0.01 and 0.05, respectively. The probability of exactly x occurrences in 75 years is then

$$P(X = x) = \left(\frac{75!}{x!(75-x)!}\right)p^{x}(1-p)^{75-x} \tag{2}$$

and the probability of one or more occurrences can be arrived at by solving 1- $P(X = 0)$. Values for the most commonly used storms are shown in Table 5-2.

Table 5-2. *Probability of Various Storms Occurring During 75-yr Design Life of Bridge*

Return period (RT in years)	**Probability**
100	0.53
500	0.13

Conditional and Combined Probabilities

Earthquakes and storms are statistically independent. Therefore, the probability of the two occurring during the design life of the bridge, but not necessarily simultaneously, is equal to the product of the individual probabilities. That is, for events A and B,

$$P(A \cap B) = P(A) * P(B) \tag{3}$$

Conditional probability is denoted as $P(A \mid B)$. This can be read as "the likelihood of a sample point in A assuming that it also belongs to B," or simply "the probability of A given B." By definition,

$$P(A \mid B) = \frac{P(A \text{ and } B)}{P(B)} \tag{4}$$

By rearranging the above expression, the probability of joint events can be expressed:

$$P(A \text{ and } B) = P(A \mid B) * P(B)$$

For combined probabilities, assume that the storm (scour) lasts two weeks and that there are 26 potential "time slots" each year. Let $P(X)$ denote the annual probability of 50-yr earthquake occurring, $P(Y)$ denote the annual probability of a 100-yr storm occurring, and $P(X \mid Y)$ denote the probability of the earthquake coinciding with the storm in one given year. Then, the annual probability of coinciding events is

$$P(X \text{ and } Y) = P(X \mid Y) * P(Y) = (1/50 * 1/26) * 1/100 = 1/130{,}000.$$

The probability of this coincidence occurring at least once during the 75-yr design life is:

$$P(N \geq 1) = 1 - P(N{=}0) = 1 - \left(1 - \frac{1}{130000}\right)^{75} = \text{1-in-1734} \tag{5}$$

Since the load factors for the Strength I traffic load combination in the AASHTO LRFD Bridge Design Specifications was calibrated to approximately 1-in-5000, it may be reasonable to design for these two coinciding events. However, if the earthquake is increased to a 150-yr event, the probability of it coinciding with the 100-yr storm is 1-in-5200. Therefore, it would not be logical to design for any more than a 150-yr earthquake coinciding with a 100-yr storm and its associated total scour depth. Long-term degradation scour, however, should be considered with any seismic event.

An Approach to Developing Hazard Tables for Owners

Collapse categories are created because the definition of failure can vary from owner to owner, from situation to situation, or as fiscal situations change. The categories below are an inspiration from the Mercalli scale for earthquakes, a system of qualitative damage description that served the engineering community quite well for many years:

I. Structure can be reopened to emergency vehicles after inspection; damage is limited to spalling of concrete cover on columns, breakage of concrete where joints have opened and closed, deformation of sacrificial or secondary members. Reinforcement has not yielded; no local buckling in steel columns, no cracking of deck, and no exposure of shear studs has occurred.
II. Girder(s) unseated, but span has not dropped due to restrainers, catchers, adequate seat length, or other "stop" mechanism.
III. One-column in a multi-column bent damaged, but still able to support dead load. Column confinement (transverse reinforcement or external 'jacket') prevents collapse.
IV. One-column in each of several multi-column bents damaged, but still able to support dead load. Column confinement (transverse reinforcement or external 'jacket') prevents collapse.
V. One pier, or, all columns in one bent are on the verge of collapse. Column confinement (transverse reinforcement or external 'jacket') prevents collapse.
VI. Two or more bents, piers, or abutments on are on the verge of dropping spans. Column confinement (transverse reinforcement or external 'jacket') prevents collapse.
VII. One dropped span; structure is repairable
VIII. One dropped span; support structure must be rebuilt
IX. Multiple spans have dropped
X. Total obliteration of the structure

For the purposes of this paper, Levels I-VI are combined into a "non-collapse" category, Levels VII-IX are combined into a "partial-collapse" category, and Level X is renamed "total collapse."

Finally the resistance of the structural system must be quantified. Although force, bending moment, and displacement are more familiar measures of member resistance, methods to equally consider all potential collapse configurations are not currently available. Thus, a vulnerability index is suggested, similar to prioritization schemes that might be used in ranking bridges for seismic retrofit funding. Each structure is given a score between 0.0 and 1.0, depending on items such as:

- Single-column vs. multi-column redundancy
- Spacing of column confinement steel
- Column heights being approximately equal
- Bent stiffnesses being approximately equal
- Adequate seat length
- Non-brittle piles in soft soil; potential for liquefaction
- Scour-critical foundations, etc.

For the purposes of this paper, however, vulnerability to collapse will simply be classified as "low," "medium," or "high." Existing structures will likely have higher vulnerabilities since structures were based on earlier knowledge and conditions.

Expanding on conditional probability calculations presented in Hida (2007), let $P(B)$ represent the likelihood of a given seismic event occurring and $P(A|B)$ represent the likelihood of a given category of collapse of a structure with a given level of vulnerability. The probability $P(A|B)$ must be based on anecdotal evidence. For example, the Loma Prieta was a M6.9 event and is known to have caused a level IX or X collapse of the Cypress Viaduct. In hindsight, we know that its vulnerability was quite high due to poor bar reinforcement detailing for joint shear and very soft soil. The Northridge earthquake, however, was a M6.7 event and caused damage between levels II and VI on structures that had been retrofitted or constructed more recently i.e. have low- to medium vulnerability. Earthquake magnitude, collapse category, and vulnerability are assumed to be correlated as shown in Table 5-3.

Table 5-3. Correlation of Collapse Level, Bridge Vulnerability Level, Earthquake Magnitude (Richter Scale)

Collapse Level Vulnerability	**Non-collapse**	**Partial Collapse**	**Total Collapse**
Low	< M7.0	M7.0 -- M8.5	> M8.5
Medium	< M5.5	M5.5 – M7.0	> M7.0
High	< M4.0	M4 – M5.5	> M5.5

To eventually calculate probabilities associated with the AASHTO LRFD Seismic Guide Specifications (publication pending), it is assumed that no events occur in what AASHTO refers to as Seismic Design Category (SDC) A. In AASHTO SDC B, C, and D, the strongest 100 events based on the Richter scale are assumed to occur as shown in Table 5-4 and as follows:

- SDC B—90% are less than M4.0; 9% are between M4 and M5.5; 1% is between M5.5 and M7.0
- SDC C--10% are less than M4.0; 85% are between M4 and M5.5; 5% are between M5.5 and M7.0
- SDC D--90% are between M4 and M5.5; 9% are between M5.5 and M7.0; 1% are greater than M7.0.

Table 5-4. Probability of Earthquake Magnitude Based on Top 100 Events in Each AASHTO Category

Magnitude (Richter)	**AASHTO SDC A**	**AASHTO SDC B**	**AASHTO SDC C**	**AASHTO SDC D**
M0.0 – M4.0	1.0	0.90	0.10	0.0
M4.0 – M5.5	0.0	0.09	0.85	0.90
M5.5 – M7.0	0.0	0.01	0.05	0.09
M7.0 – M8.5	0.0	0.0	0.0	0.01

By combining the information in Tables 5-3 and 5-4, the distribution of the strongest 100 events in each collapse category for each vulnerability level can be displayed as shown in Table 5-5. For example, a bridge with 'high' vulnerability in SDC C will be standing, partially collapsed, or totally collapsed 10, 85, and 5% of the time, respectively, based on the strongest 100 events recorded.

Tables 5-5a, 5-5b, 5-5c. Probabilities of Strongest 100 Events in Collapse Categories for Each Vulnerability Level

SDC B	**Collapse Level Vulnerability**	**Non-collapse**	**Partial Collapse**	**Total Collapse**
	Low	1.00	0.00	0.00
	Medium	0.99	0.01	0.00
	High	0.90	0.09	0.01

SDC C	**Collapse Level Vulnerability**	**Non-collapse**	**Partial Collapse**	**Total Collapse**
	Low	1.00	0.00	0.00
	Medium	0.95	0.05	0.00
	High	0.10	0.85	0.05

SDC D	**Collapse Level Vulnerability**	**Non-collapse**	**Partial Collapse**	**Total Collapse**
	Low	0.99	0.01	0.00
	Medium	0.90	0.09	0.01
	High	0.0	0.90	0.10

The likelihoods of seismic events with various recurrence intervals occurring were given in Table 5-1. It must be noted, though, that labeling a fault with one recurrence level is troublesome from the geo-science standpoint because all "slip" does not take place in one event. Hence, Table 5-6 generalizes events into 'frequent,' 'infrequent,' and 'rare' and assumes very conservative annual probabilities of occurrence.

Table 5-6. Generalized Earthquake Return and Probabilities

Simplified EQ Return	**Assumed Annual Probability**
Frequent	0.50
Infrequent	0.30
Rare	0.05

Conditional and combined probabilities are calculated where M denotes a degree of collapse for a given degree of vulnerability and seismic design category, and N denotes the occurrence of an event. For example, the probability of 'partial collapse' of a structure in SDC C with 'high' vulnerability that is subjected to an 'infrequent' event is calculated as follows:

$P(M \text{ and } N)=P(M \mid N)*P(N) = 0.85*0.30 = 0.255 \Rightarrow$ 1 in 3.9 chance of occurrence. If the vulnerability could be improved to 'medium', these values would improve to 1 in 67. This and other cases are listed in Tables 5-7 and 5-8.

Tables 5-7a, 5-7b, 5-7c. Annual Probabilities of Collapse Categories for each Vulnerability Level

SDC B	**Frequent Event Collapse Level**			**Infrequent Event Collapse Level**			**Rare Event Collapse Level**		
Vulnerability	**Non-**	**Part-**	**Total**	**Non-**	**Part-**	**Total**	**Non-**	**Part-**	**Total**
Low	0.500	0.000	0.000	0.30	0.000	0.000	0.050	0.000	0.000
Medium	0.495	0.005	0.000	0.297	0.003	0.000	0.049	0.001	0.000
High	0.450	0.045	0.005	0.270	0.027	0.003	0.045	0.005	0.001

SDC C	**Frequent Event Collapse Level**			**Infrequent Event Collapse Level**			**Rare Event Collapse Level**		
Vulnerability	**Non-**	**Part-**	**Total**	**Non-**	**Part-**	**Total**	**Non-**	**Part-**	**Total**
Low	0.500	0.000	0.000	0.300	0.000	0.000	0.050	0.000	0.000
Medium	0.475	0.025	0.000	0.285	0.015	0.000	0.048	0.003	0.000
High	0.050	0.425	0.025	0.030	0.255	0.150	0.005	0.043	0.003

SDC D	**Frequent Event Collapse Level**			**Infrequent Event Collapse Level**			**Rare Event Collapse Level**		
Vulnerability	**Non-**	**Part-**	**Total**	**Non-**	**Part-**	**Total**	**Non-**	**Part-**	**Total**
Low	0.495	0.005	0.000	0.297	0.003	0.000	0.050	0.001	0.000
Medium	0.450	0.045	0.005	0.270	0.027	0.003	0.045	0.005	0.001
High	0.000	0.450	0.050	0.000	0.270	0.030	0.000	0.045	0.005

Table 5-8a, 5-8b, 5-8c. Hazard "1-in-X"

SDC B	**Frequent Event Collapse Level**			**Infrequent Event Collapse Level**			**Rare Event Collapse Level**		
Vulnerability	**Non-**	**Part-**	**Total**	**Non-**	**Part-**	**Total**	**Non-**	**Part-**	**Total**
Low	2	--	--	3.3	--	--	20	--	--
Medium	2	200	--	3.4	333	--	20	2000	--
High	2.2	22	200	3.7	37	333	22	222	2000

SDC C	**Frequent Event Collapse Level**			**Infrequent Event Collapse Level**			**Rare Event Collapse Level**		
Vulnerability	**Non-**	**Part-**	**Total**	**Non-**	**Part-**	**Total**	**Non-**	**Part-**	**Total**
Low	2	--	--	3.3	--	--	20	--	--
Medium	2.1	40	--	3.5	67	--	21	400	--
High	20	2.4	40	33	3.9	67	200	24	400

SDC D	**Frequent Event Collapse Level**			**Infrequent Event Collapse Level**			**Rare Event Collapse Level**		
Vulnerability	**Non-**	**Part-**	**Total**	**Non-**	**Part-**	**Total**	**Non-**	**Part-**	**Total**
Low	2	200	--	3.4	333	--	20	20	--
Medium	2.2	22	200	3.7	37	333	22	222	2000
High	--	2.2	20	--	3.7	33	--	22	200

Discussion

The AASHTO LRFD Seismic Guide Specifications require that bridges be designed for life safety unless an operational objective is established and authorized by the owner. In terms of the collapse levels defined herein, most bridges in the United States will then be designed to non-collapse levels II-VI with selected structures designed to collapse level I. New structures will have lower vulnerabilities and retrofit of existing structures will strive to improve the vulnerability and bring the structure into collapse levels II-VI. The author recognizes that other countries and cultures may strive for serviceability i.e. collapse level I.

The intent of this paper is to be of assistance for U.S. projects early in the planning and budgeting stages. The added cost to bridge owners for developing plastic hinges or confining column steel, permitting fusing, restraining girders, is justified in terms of risk. [For seismic retrofit, different strategies are likely to have more variability in outcomes and cost than new construction.] Furthermore:

- An owner concerned about lowering construction costs can choose to accept more damage. On the other hand, the cost may be perceived as negligible for the level of safety to be gained. [A study by Ketchum (2004) showed that a 10% increase in peak ground acceleration on high bridges in high seismic zones, was associated with a 10 to 12% increase in construction cost.]
- An owner questioning the need to design for earthquake in areas of infrequent occurrence can be told the risks.
- An owner questioning the need for seismic retrofit can see the difference in risk and performance between high- and medium-vulnerability structures.
- An owner demanding an “earthquake-proof” structure can be softened with the menu of “collapse categories” herein.
- An owner with limited resources can weigh the risks of various projects in a jurisdiction.
- An owner who isn’t an engineer but wants guidance in making public works decisions, can be given the risk descriptions herein. For a broader perspective, comparisons can be made to other public works such as nuclear power plants and dams.
- An owner concerned about NOT designing for simultaneous hazards can be placated with terminology akin to comparing the annual risk of automobile accident to that of an airplane crash (say, 1 in 1000 vs. 1 in 10,000).

Although the hazard tables presented in this paper are rough and based on very simple concepts, the load and resistance are considered together. In the context of probability-based limit state design or evaluation, this feature is *essential*. In other words, a meaningful safety evaluation depends on what structure is in question and what level of damage is acceptable--in addition to what magnitude event is appropriate for design.

Summary

Methodologies have been described to: (1) justify not combining force effects from more than a 150-yr earthquake and a 100-yr storm for bridge design; and (2) develop probability-based seismic hazard tables for bridge owners and bridge stakeholders. The processes could be extended to wave-force related damage, ship collision scenarios, or various blast events. Interpolation and engineering judgment may be required in order to correlate vulnerability to collapse when information on past events is limited. Nevertheless, such tables can be vital in commencing discussions on less common loads, associated risk, and the cost-benefits in reducing vulnerability. Since the recent collapse of the I-35 bridge in Minnesota, the public has become more concerned for our aging infrastructure and interested in budgets for public works projects. Engineers must be able to comparatively express risk to structures of various levels of vulnerability during catastrophic events using risk terminology that the general public understands. Engineers must similarly be able to defend themselves for not designing for less common load combinations.

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Extreme Hazard Quantifications from Hindsight to Forecast

6

John M. Kulicki

In 2004, Hurricane Ivan caused significant damage to numerous structures along the northwest coast of Florida. One of the most costly was the damage to the I-10 Bridges over Escambia Bay. The I-10 Bridges over Escambia Bay were similar to many low-lying coastal bridges composed of numerous relatively short spans located close to the mean high water level. These types of structures are susceptible to extreme horizontal and vertical forces and moments exerted by waves brought within reach of the superstructure by storm surge. In the case of the I-10 Bridges over Escambia Bay, the combination of elevated water level and wave heights trapped air between the girders increasing the buoyant force and imparted large vertical and horizontal forces dislodging most of the low lying spans and in some cases moving the superstructures some distance from their original piers. In 2005, Hurricane Katrina in particular, and to a lesser degree Hurricane Rita, did similar damage to bridges in Mississippi and Louisiana. The low-lying bridges in the Biloxi Bay area, particularly those along US-90, received similar types of damage to bridge superstructures, largely associated with the superstructures being forced free from the substructure through a combination of vertical and horizontal hydraulic forces.

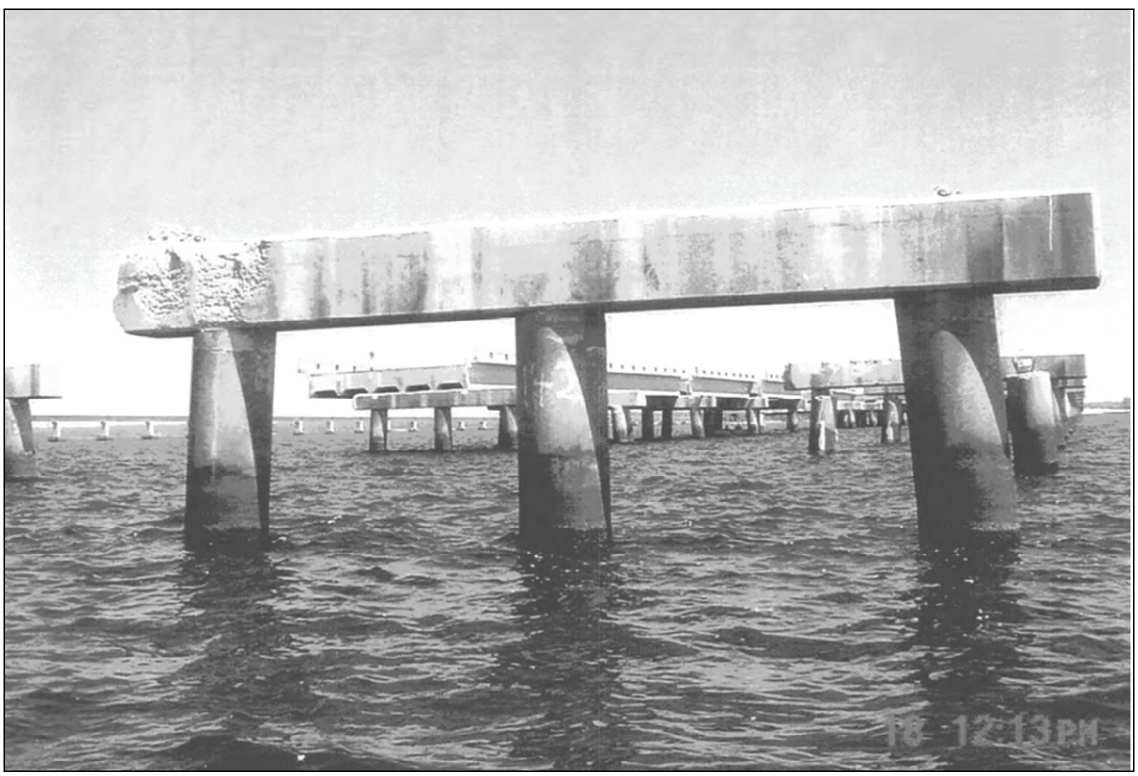
Similar damage was noted on the I-10 Twin Spans across Lake Pontchartrain north of New Orleans. Once again, the combined action of elevated water level and wave forces dislodged many spans. Some of the spans were dislodged but remained on the substructure; many were completely dislodged and collapsed. Figures 6-1 through 6-3 show the type of damage inflicted on these bridges.

Ghara, 2006

Figure 6-1. Dislodged Spans of I-10 Twin Spans, Lake Pontchartrain, Louisiana, Hurricane Katrina

Photo courtesy of Mississippi DOT

Figure 6-2. Dislodged Spans, U.S. 90 over Bay St. Louis, Mississippi, Katrina



Florida DOT, 2004

Figure 6-3. Collapsed Spans, I-10 over Escambia Bay, Florida, Hurricane Ivan

It is of some interest to note that similar levels of damage were noted on bridges as a result of Hurricane Camille in 1969 (Barksdale, 1970). At that time, the combined action of buoyancy from entrapped air, as well as the action of wave forces, to dislodge spans, were identified by staff of the Mississippi Department of Transportation.

In response to the need to learn more about how to design and retrofit coastal bridges subjected to these types of loads, the FHWA has authorized work under Task Order DTFH61-06-T-70006, Development of Guide Specifications and Handbook Retrofit Options for bridges vulnerable to coastal storms. A team led by Modjeski and Masters, Inc., but including coastal engineers Moffatt & Nichol, Inc. and Ocean Engineering Associates, Inc (OEA), geotechnical experts from D'Appolonia, Inc. and assistance from Professor Dennis R. Mertz was selected for the project. A summary of the tasks given to this group include:

- Task 1 - A series of project meetings
- Task 2 – Review, summarize and augment literature
- Task 3 – Review and supplement ongoing wave force studies
- Task 4 – Compile and catalog retrofit options
- Task 5 – Perform a study of A retrofit option
- Task 6 – Develop a Guide Specification for adoption by AASHTO and a retrofit handbook.
- Task 7 – Complete a project report
- Task 8 – Prepare information packages

As of this writing Tasks 1, 2, 3 and 4 are complete, Task 5 is essentially complete and Task 6 is in an advanced stage of development. Task 6 is clearly the core deliverable and a 50% specification and manual were submitted in May 2007. The engineers from North Carolina, with help from Moffatt & Nichol, Inc. did a "test drive" of the 50% specifications and made numerous comments on organization and suggestions for clarifications that were quite valuable. The Specification and Retrofit Handbook are to be 90% complete by the end of August 2007. The target is to have a Guide Specification for consideration for adoption at the 2008 meeting of AASHTO's Highway Subcommittee on Bridges and Structures.

The literature study undertaken in Task 2 showed that while many investigators have studied the action of waves on structures, most of the work concentrated on the vertical components of the structures or structures that differ greatly from a bridge superstructure. Furthermore, the existing research on superstructure components concentrate on offshore environments, where the design wavelength is much greater than the length of the structure. Conversely, bridges typically span inland waterways where the wavelengths are shorter (as compared to the bridge superstructure widths). The wave-induced hydrodynamics (velocities and accelerations) of the two situations are quite different. When the wavelength is much greater than the width of the superstructure, the components of the wave-induced hydrodynamic have much smaller variations over the structure at any point in time. This is in contrast with the large variations in force over the structure that exist for shorter wave lengths After a review of the available studies, four wave force prediction methods were selected as being potentially applicable to bridge structures. They are:

- The process developed at H.R. Wallingford in Great Britain leading to an exponential formulation of the wave force equation for quasi-static vertical forces (McConnell et al., 2004).
- A refinement from H.R. Wallingford involving a linear equation (Alsop et al., 2005).
- An approximation of the Wallingford method by Douglas (Douglas et al., 2006).
- OEA's Physics Based Model (PBM), an extension and modification of Morrisons equation following Kaplan's adaptation to horizontal structures (Kaplan et al., 1995).

After some review, the Wallingford exponential method was eliminated from further consideration because the Wallingford linear method was a new evolution thereof. Initial comparisons of calculated forces resulted in elimination of the Douglas method because it was generally more conservative than the Wallingford method due to simplifications that had been made. Both the Douglass and Wallingford methods were not intended for application to submerged structures. The PBM which can be applied to both submerged and subaerial structures also had greater appeal due to its theoretical completeness.

Physical model tests conducted in a 6 ft wide, 6 ft deep, 130 ft long wave tank at the University of Florida were used to establish drag and inertia coefficients and to refine the PBM. Data from these tests were also used to compare the various predictive equations. Figure 6-4 shows one test in the wave tank. The impingement of the wave on the right side of the photograph is clearly evident, as is the amount of retained (entrapped) air in the cells of the superstructure. This bridge was a model that had the characteristics of one span of the I-10 Bridges over Escambia Bay.

Numerous plots were developed showing the comparison of wave forces predicted by various methods and the results of the wave tank experiments by OEA. Figure 6-5 shows one such comparison illustrating the quasi-static vertical force computed by the Wallingford exponential method and laboratory results from the wave tank without a slamming force.

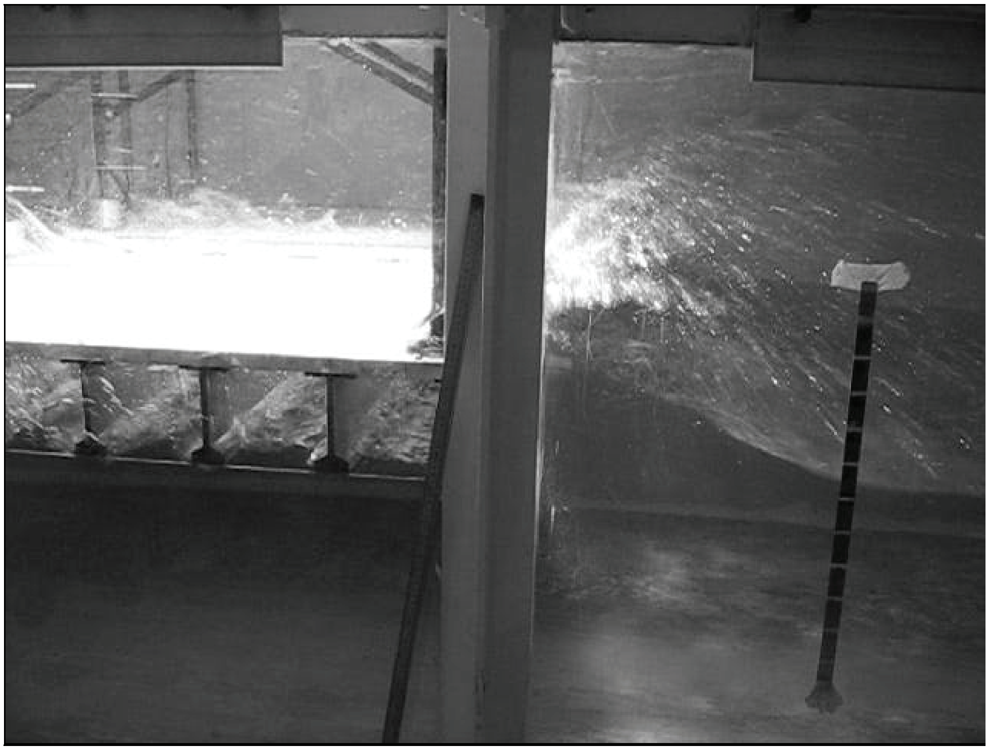


Photo courtesy of J. M. Kulicki

Figure 6-4. Bridge Model in Wave Tank at University of Florida

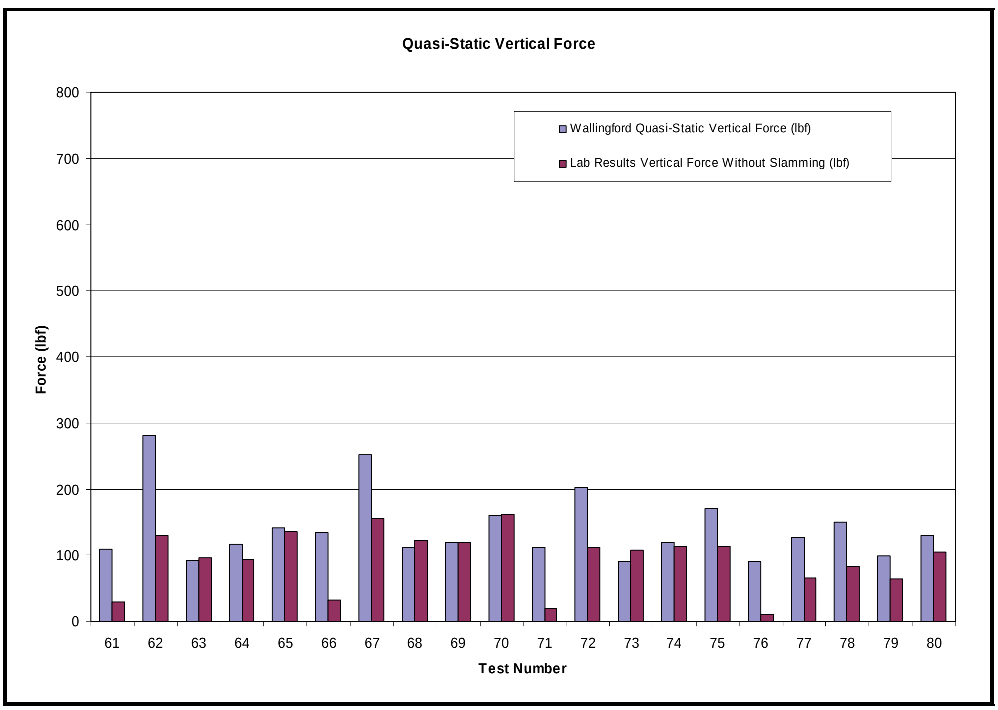


Figure 6-5. Preliminary Compassion of the Wallingford Exponential Method and Measured Vertical Force
(Calculated results by Moffatt & Nichol, Inc. measured values by University of Florida and OEA.)

The basis for a choice of a method to be proposed in the Guide Specification involved evaluation of the predicted forces compared to the physical model test data, the prediction of failures on the I10-Escambia Bay Bridges, theoretical completeness, and practicality. Based on an evaluation of all these factors, it was decided to proceed with the PBM method.

It was not practical from both the time and resources available on this project to conduct physical model tests for a wide range of bridge span widths, girder heights, and water and wave conditions thus the following methodology was proposed and implemented by OEA. Once the PBM was developed and the drag and inertia coefficients determined from the physical model tests refinements were made to the model until satisfactory agreement was reached between model predictions and physical model test results. The PBM was then run for a range of span configurations and elevations (relative to the storm water level), water depths, and wave conditions and the maximum vertical force and associated horizontal force and moment data compiled. This data was then used to develop parametric equations for maximum vertical force, associated horizontal force and associated moment in terms of the pertinent dimensionless groups involving structure, water and wave variables.

One of the comparisons between the PBM prediction and measured quasi-static vertical force is shown in Figure 6-6. Not all predictions were as good as this one but all were considered acceptable.

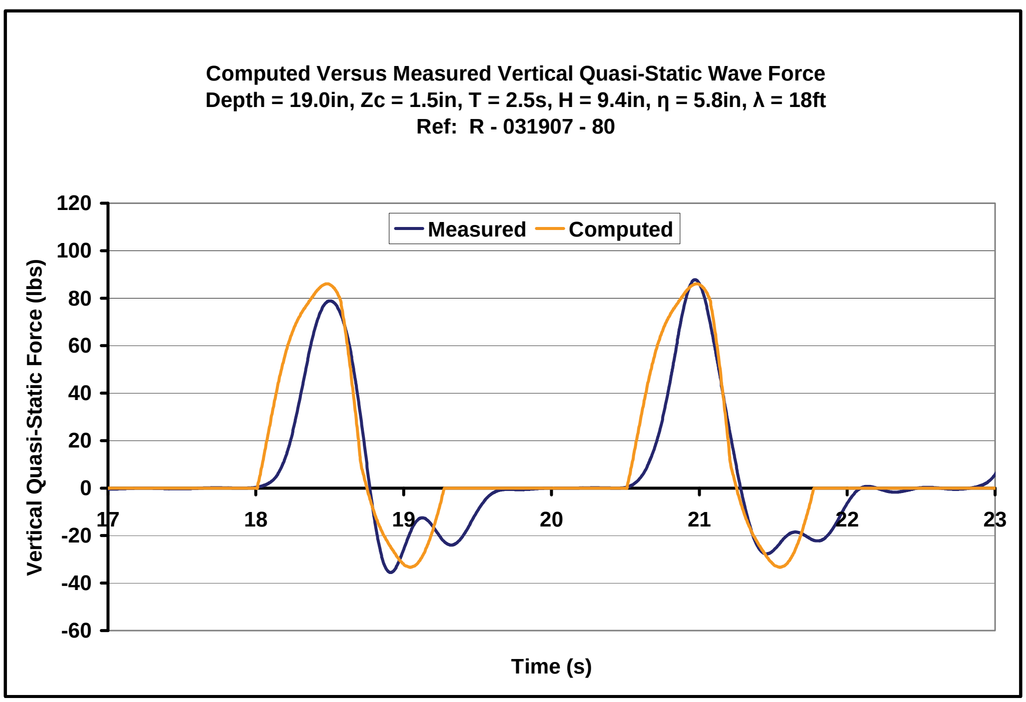


Figure 6-6. Comparison of Predicted and Measured Quasi-static Vertical Forces for One Set of Parameters

Going from a laboratory results to a field situation involves many parameters, some of which are clearly multi-hazard situations. A typical list of variables necessary to complete an analysis of a given bridge include:

- The bridge location in the body of water.
- Bathymetry to characterize the depth of the body of water.
- Fetch length and orientation.
- Design wave height and design wave period.
- Design wind velocity.
- Design water surface elevation composed of astronomical tide, storm surge, and local wind setup/set down – All a multi-hazard situation.
- Design current velocity.
- Span dimensions, shape, and low member height above storm water level.

A three-level approach is being adopted in order to analyze the wave forces for a given situation based on 100 year design conditions.

Level 1 involves a conservative evaluation of the situation using "available data" for wind and storm water height. The appropriate sources for data for the Level 1 analysis would be either site-specific information on astronomical tide, wind and surge, or the use of the ASCE 7-05 wind maps adjusted for 100 year return period in combination with FEMA storm surge maps (ASCE, 2005). In some cases the FEMA surge data contains the astronomical tide, and the engineer needs to know if this is the case. Approximate wave conditions (significant wave heights and peak periods) sufficient for a Level I analysis can be obtained through the use of empirical equations located in the U.S. Army Corps of Engineers Shore Protection Manual and its replacement, The Coastal Engineering Manual (U.S. Army, 1984, 2002). The equations for significant wave height (average of the largest 1/3 wave heights in a Rayleigh distribution of random waves in a given storm) and wave period are shown below. Equations are also available in the Draft Guide Specifications to estimate local wind setup and other wave characteristics. They are omitted here for required brevity.

$$T_p = 7.54 \tanh\left[0.833\left(\frac{gd}{U_A^{\ 2}}\right)^{3/8}\right] \tanh\left\{\frac{0.0379\left(\frac{gF}{U_A^{\ 2}}\right)^{1/3}}{\tanh\left[0.833\left(\frac{gd}{U_A^{\ 2}}\right)^{3/8}\right]}\right\}\left(\frac{U_A}{g}\right) \tag{1}$$

$$H_s = 0.283 \tanh\left[0.53\left(\frac{gd}{U_A^2}\right)^{3/4}\right]\tanh\left\{\frac{0.00565\left(\frac{gF}{U_A^2}\right)^{1/2}}{\tanh\left[0.53\left(\frac{gd}{U_A^2}\right)^{3/4}\right]}\right\}\left(\frac{U_A^2}{g}\right) \qquad (2)$$

Where:

g = gravitational constant (ft/sec^2)
U_A = wind-stress factor (ft/sec)
d = average water depth over the fetch including surge, astronomical tide, and local wind setup (ft)
F = fetch length in the direction of the wind from the upwind shore (ft)
T_p = wave period (sec)
t = duration of U_s (sec)

There are a series of corrections that are possible for the location of the bridge in the body of water, as well as adjustments for the duration of wind necessary to make a fetch limited wave in a particular body of water with the wind coming from a particular direction. In the Level 1 analysis, all these parameters are assumed to be maximized at the same time, clearly a conservative approximation for most cases. Due to the fact that many structural engineers are unfamiliar with the data required to perform even a Level 1 analysis, a review of the input data and the results of the analysis by a qualified coastal engineer is recommended.

A Level 3 analysis is much more thorough and involves the use of a number of sophisticated storm surge and wave computer models and must be performed by qualified coastal engineers. The typical steps involved would be:

- Perform hurricane wind and pressure field hindcast for as many hurricanes (or significant storms) as have impacted the area of interest as time and resources allow.
- Perform storm surge and wave hindcasts using coupled wave and surge models for hurricanes using the hindcasted wind and pressure fields.
- Using the water elevation and wave information at the bridge site for each of the hindcasted storms, perform extremal analyses on these parameters to obtain the desired design water levels and wave conditions.
- Knowing the design water levels, wave conditions and structure parameters compute the storm surge and wave induced forces and moments on the bridge spans.

Level 3 analysis is clearly the realm of qualified coastal engineers.

A Level 2 analysis falls somewhere between Level 1 and Level 3 analyses and considerable latitude is permitted in the choice of how far between Level 1 and Level 3 the engineer believes is appropriate for the particular situation. Level 2 may be used to improve upon any of the data or analytic techniques used in Equation 1. It allows the designer to make an engineering judgment as to which design parameters have the least reliability, both with respect to their absolute magnitudes as well as a joint probability of occurrence with the other parameters and, therefore, warrant further analysis. Level 2 should employ wave modeling (in place of the empirical Corps of Engineers equations) and nonlinear wave theory to improve the accuracy of the design wave parameters.

In summary, the reported research has resulted in a major step forward in characterizing the forces on bridges caused by storm surge and waves. The design water level is composed primarily of storm surge and local wind setup. These are, for the most part, independent quantities. The wave conditions at the site depend on several quantities including the wind speed and duration, the fetch, water depth, etc. The wave conditions are dependent on the storm surge (through the water depth) and the local wind setup since both are driven by the wind. This lack of independence of these quantities greatly complicates joint probability analyses. The proposed design specification will address this problem by specifying a multilevel analysis approach. The lowest level of analysis, assumes that all contributors to wave crest elevation are maximized simultaneously. The proposed design specifications will recognize that advanced storm and wave modeling can produce significant improvements compared to empirical equations such as Equations 1 and 2 herein, and can address joint probability for a particular hindcasted storm, i.e. a storm based on measurements of a past event.

Acknowledgements

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Recent Adoption of the 2007 LRFD Guide Specifications for Seismic Design of Bridges

7

Roy A. Imbsen

Introduction and Background

The work that led to the adoption of the AASHTO LRFD Guide Specification for Seismic Design of Highway Bridges began with the development of the AASHTO LRFD Bridge Design Specifications over a decade ago. Recent major earthquakes that have occurred in the Western United States, Japan, Taiwan and Turkey have increased public awareness of the losses that occur as a result of earthquakes serving as a reminder to the engineering profession that modern day design specifications are needed.

The NCHRP Project 12-49 "Recommended LRFD Guidelines for the Seismic Design of Highway Bridges." Part 1: Specifications were prepared as part of the overall NCHRP Project "Comprehensive Specifications for the Seismic Design of Bridges." The Guidelines were completed and submitted to the AASHTO Bridge Engineers for review. Following an evaluation by several states, the Bridge Engineers did not accept the Guidelines as written and recommended that some changes be made, which would make the document more useable by the practicing bridge engineer. The evaluation included trial designs from the states of: New Jersey, Washington, Arkansas, Oregon, South Carolina. Tennessee and Nevada. Additionally, since the NCHRP Project 12-49 was completed, other new developments had surfaced, which were then considered as updates to be included in the follow-up NCHRP 20-07/193 Recommended Guidelines for Seismic Design of Bridges. The second NCHRP effort coupled with the assistance and support of the AASHTO T-3 Committee for Seismic Design resulted in the AASHTO adoption of this second NCHRP as the AASHTO Guide Specifications for Seismic Design of Bridges at the 2007 meeting in Delaware.

The overall effort leading to the final adoption included several tasks which were recommended by AASHTO T-3 Committee. Initially Task F3-4 was initiated in order to assist AASHTO's T-3 Subcommittee for Seismic Design in finding a consensus among state bridge engineers on what should be adopted as seismic guidelines for the LRFD design of bridges. The task was not to produce a document to be adopted but to layout a "road map" for development of the document. The screening and retention of appropriate material targeted for use in the development of the new LRFD Guidelines was conducted taking into consideration the following aspects:

1. Research conducted since AASHTO'S Division 1-A

2. Ensuring practical and implementable guidelines
3. Cost effective use of public funds
4. Reasonable tolerance of risk
5. Ease of use by a practicing bridge engineer
6. Review of comments explaining the reasons why NCHRP 12-49 was not accepted by AASHTO T-3 Committee along with other related reference documents
7. Ensuring that the guidelines are adoptable by AASHTO (based on discussion with T-3 Committee Members and other states)

The objective of Task F3-4 was to propose a road map that draws special consideration to the issues highlighted by the AASHTO's T-3 Committee needing improvement or modification. These issues were identified as follows:

1. Selection of Return Period and Design Spectrum for a Single Hazard Level, which is more consistent with the hazards used in bridge design addressing a No Collapse Criteria of bridges.
2. Range of Applicability for No Analysis or Limited Analysis.
3. Implementation of a User Friendly Proposed Criteria.

Achieving consensus on these issues was considered a major milestone in the road towards the adoption of seismic guidelines for the LRFD design of bridges.

Recommended Approach to Addressing Seismic Hazard

The AASHTO Guide Specifications for LRFD Seismic Bridge Design is documented in the NCHRP 20-07/Task 193 Task 6 Report. Task 6 contains a section addressing seismic hazard in accordance to the road map established in the above mentioned Task F3-4. The recommended approach to addressing the seismic hazard is based on the following positions:

- Recommendations would be Primarily for Design against the Effects Ground Shaking Hazard
- Selection of a Return Period for Design less than 2500 Years
- Inclusion of the USGS 2002 Update of the National Seismic Hazard Maps
- Effects of Near Field and Fault Rupture to be addressed separately
- Displacement Based Approach with both Design Spectral Acceleration and corresponding Displacement Spectra provided
- Hazard Map under the control of AASHTO with each State having the option to Modify or Update their own State Hazard using the most recent Seismological Studies consistent with the Established Risk

Background on Seismic Hazard

The current State of the Practice in addressing the seismic hazard for the design of bridges in the U.S. has evolved from just conforming to AASHTO Division 1-A requirements to adopting higher standards that take into account the possible effects of larger earthquakes in the Eastern United States and the impacts of major earthquakes that occurred recently in the Western United States, Japan, Taiwan and Turkey. This change in the Seismic Hazard Practice can be best illustrated in looking at the following sources:

- NEHRP 1997 Seismic Hazard Practice
- Caltrans Seismic Hazard Practice
- NYCDOT and NYSDOT Seismic Hazard Practice
- NCHRP 12-49 Seismic Hazard Practice
- SCDOT Seismic Hazard Practice
- Site Specific Hazard Analyses Conducted for Critical Bridges

Appendix 2A of the NCHRP 20-07/Task 193 Task 6 Report contains background on seismic hazard drawn from the above mentioned sources.

Proposed Seismic Hazard for Design of Normal Bridges

In reviewing the seismic hazard practice in different regions as described previously, it was apparent that some important aspects of this Practice need to be taken into consideration when developing new Guidelines. These aspects were pivotal in reaching the objective of producing Guidelines that are adoptable by AASHTO.

These aspects include:

1. Consideration for lower return period for Design based on the Maximum Considered Earthquake (MCE) maps published in 1996 with USGS 2002 Update shall be considered a minimum standard. Modification or increase in the hazard intensity based on Seismological Studies needs to be included as an option for states and agencies seeking a higher degree of hazard identification to a specific region or bridge.
2. The reduction in the design intensity can be implicitly achieved by considering applying a reduction/modifier factor for design spectrum derived from USGS MCE maps. An alternative to this approach would be embarking on developing new maps based on a modified new definition of the MCE for Bridge Design.
3. Consideration of applying a reduction factor on the hazard intensity for existing bridges or bridges located in rural areas.
4. Selection of a lower return period for Design is made such that Collapse Prevention is not compromised when considering historical large earthquakes. This reduction can be achieved by taking advantage of sources of conservatism not explicitly taken into account in current design procedures. These sources of conservatism are becoming obvious based on recent findings from both observations of earthquake damage and experimental data. Some of these sources are shown in Table 7-1.

Table 7-1. Identified Sources of Conservatism

Source of Conservatism	Safety Factor
Computational vs. Experimental Displacement Capacity of Components	1.3
Effective Damping	1.2 to 1.5
Dynamic Effect (i.e., strain rate effect)	1.2
Pushover Techniques Governed by First Plastic Hinge to Reach Ultimate Capacity	1.2 to 1.5
Out of Phase Displacement at Hinge Seat	Addressed in Task 6 Report

The conservatism is directly coupled to the seismic reliability of the structural system under consideration. The current state of the practice favors continuous superstructures for the majority of bridges with an objective of minimizing expansion joints to gain functionality, reduce maintenance, and increase life cycle of the bridge. This selection has a favorable impact on the earthquake redundancy of the bridge system. Considering a single performance level of "No Collapse", the seismic redundancy of the bridge system is enhanced with the increase of the number of plastic hinges that must yield and then fail in order to produce the impending collapse of the structure. This enhanced redundancy translates into a delayed failure (i.e. collapse) provided sufficient seat width exists in the bridge system. Therefore two distinctly different aspects of the design process need to be provided:

a. An appropriate method to design adequate seat width(s) considering out of phase motions.
b. An appropriate method to design the ductile substructure components without undue conservatism.

These two aspects are embedded with different levels of conservatism that need to be calibrated against the single level of hazard considered in the design process.

The first aspect is highly influenced by variation in the periods of the frames on both sides of a joint as well as the damping generated by the ductile behavior of plastic hinges. This aspect is addressed in terms of recommendations or limits on periods ratio for frames on both sides of an expansion joint.

The second aspect is addressed using a static push-over analysis. As shown in Figure 7-1, the collapse displacement is usually reached when the P-Δ line intersects the load-displacement curve of the structure, because at this point, any increment in displacement produces an increment in the P-Δ effect due to gravity loads that cannot be resisted by the lateral resistant system. It is important to mention that for structures with relatively small gravity loads, a much larger reduction in component strength can be tolerated without reaching structural collapse. This is especially relevant to bridge columns carrying relatively low axial loads. In essence, the continuity of the superstructure and low axial loads in columns make a typical bridge more resilient against collapse in a seismic event.

Under earthquake ground motions at the supports, the structure or any of its components can fail under a smaller displacement than the displacement $\Delta_{collapse}$ illustrated in Figure 7-1. This failure is mainly attributed to nonsymmetric cumulative plastic displacement that is highly dependent on the characteristics of the earthquake ground motions. The reliable displacement capacity is typically associated with the displacement corresponding to a limited decrease in strength of 20% to 30% maximum, obtained under monotonically increasing deformation. As shown in Figure 7-1, the displacement capacity $\Delta_{capacity}$ can only be established given the descending slope following the point of maximum lateral resistance F_{max}. Recognizing the complexity of determining $\Delta_{capacity}$, the $\Delta_{capacity/bridge}$ is used as a conservative and simple measure assuming nominal properties.

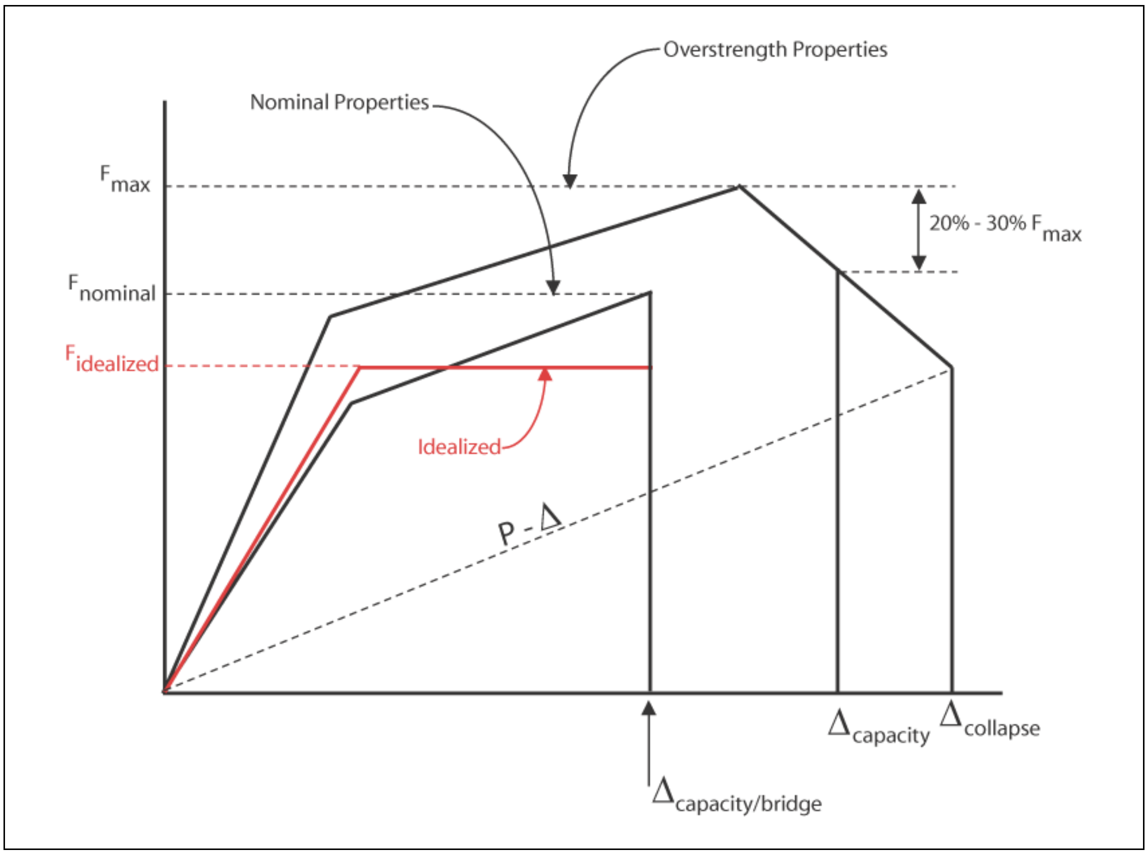

Figure 7-1. Idealized Load – Deflection Curve of a Bridge

In summary, the two aspects described above should be considered in the practice to justify a reduction in the design hazard and ensure the development of a simplified methodology that addresses the different sources of conservatism included in the current state of the practice.

In order to assess the feasibility of a reduction in hazard from the 2% in 50 years hazard level adopted by NCHRP 12-49, a Probabilistic/Deterministic comparison is conducted on 20 sites. Table 7-2 shows the state, city, dominant source, latitude and longitude of the selected sites. Table 7-3 shows the short period and one-second spectral acceleration for the Deterministic Seismic Hazard Analysis (DSHA), the Deterministic Cap taken at 1.5 times the (DSHA) value, the Maximum Considered Earthquake, and the Design Spectral Acceleration S_{DS} and S_{D1} based on NEHRP 1997 guidelines.

Table 7-2. Selected Sites for PSHA/DSHA Comparison

ST	CITY	FEATURE	DOMINANT SOURCE	LATITUDE	LONGITUDE
CA	Daly City	Zip Code 94015	San Andreas	37.681240	-122.479000
CA	San Francisco	City Hall	San Andreas	37.779083	-122.417450
CA	SFOBB	Site from Po/Roy	Hayward	37.750000	-122.250000
CA	Berkeley	Site from Po/Roy	Hayward	37.871667	-122.271667
CA	Benicia Martinez	Site from Po/Roy	Concord	38.000000	-122.116667
CA	Los Angeles	City Hall	Puente Hills blind thrust	34.053700	-118.243183
CA	Vincent Thomas	Site from Po/Roy Location corrected	Palos Verdes	33.749218	-118.271466
CA	Long Beach	Zip Code 90810	Newport-Inglewood	33.813890	-118.217000
CA	Coronado Bridge	Site from Po/Roy	Rose Canyon	32.616667	-117.116667
WA	Seattle	Space Needle	Seattle fault zone	47.621150	-122.348950
WA	Tacoma North	Site from Po/Roy	Seattle fault zone	47.250000	-122.366667
UT	Salt Lake City	State Capital	Wasatch fault, Salt Lake City section	40.776367	-111.887983
UT	Salt Lake City	Site from Po/Roy	Wasatch fault, Salt Lake City section	40.750000	-111.883333
IN	Evansville	Zip code 47720	New Madrid fault zone	38.023280	-87.617100
MO	St. Louis	Zip code 63129	New Madrid fault zone	38.466780	-90.319400
KY	Paducah	Zip code 42003	New Madrid fault zone	37.034190	-88.603800
TN	Union City	Zip code 38261	New Madrid fault zone	36.428110	-89.059500
TN	Memphis	City Hall	New Madrid fault zone	35.148750	-90.054700
TN	Memphis	Zip code 38127	New Madrid fault zone	35.225170	-90.008400

Table 7-3. Design Spectral Acceleration based on NCHRP 1997

CITY	Det Ss, g	Det S1, g	1.5*Det Ss, g	1.5*Det S1, g	MCE Ss, g	MCE S1, g	SDs, g	SD1, g
Daly City	1.49	0.85	2.23	1.28	2.23	1.28	1.49	0.85
San Francisco	0.88	0.44	1.32	0.67	1.50	0.67	1.00	0.44
SFOBB	0.84	0.30	1.26	0.45	1.50	0.60	1.00	0.40
Berkeley	1.28	0.49	1.93	0.74	1.93	0.74	1.28	0.49
Benicia Martinez	0.96	0.31	1.44	0.47	1.50	0.60	1.00	0.40
Los Angeles	1.50	0.57	2.24	0.86	2.19	0.74	1.46	0.49
Vincent Thomas	1.41	0.63	2.12	0.95	2.08	0.92	1.38	0.61
Long Beach	1.28	0.51	1.91	0.77	1.81	0.70	1.20	0.47
Coronado Bridge	1.19	0.47	1.78	0.70	1.37	0.54	0.91	0.36
Seattle	1.34	0.48	2.01	0.73	1.41	0.48	0.94	0.32
Tacoma North	0.47	0.18	0.71	0.28	1.20	0.41	0.80	0.27
Salt Lake City	1.28	0.53	1.92	0.80	1.71	0.69	1.14	0.46
Salt Lake City	1.25	0.53	1.88	0.79	1.70	0.69	1.13	0.46
Evansville	0.27	0.09	0.41	0.13	0.67	0.19	0.45	0.13
St. Louis	0.23	0.08	0.34	0.12	0.61	0.17	0.40	0.12
Paducah	0.89	0.24	1.33	0.36	1.50	0.47	1.00	0.31
Union City	0.86	0.23	1.29	0.35	1.50	0.57	1.00	0.38
Memphis	0.60	0.17	0.91	0.25	1.40	0.38	0.93	0.25
Memphis	0.65	0.18	0.98	0.27	1.50	0.41	1.00	0.27

Table 7-4 shows the short period and one-second acceleration based on a Probabilistic Seismic Hazard Analysis (PSHA) for 10% and 5% exceedance in 50 years. Table 7-4 includes two additional sites to the 20 sites identified in Table 7-2 and 7-3.

Table 7-5 shows the short period and one-second period acceleration including Type D soil effect for the proposed 5% exceedance in 50 years Design Spectrum.

Table 7-6 shows a comparison of the one-second acceleration (PSHA) to the USGS 1996. As seen from Table 7-6, California sites show a decrease of the acceleration values while other sites show a marginal change or an increase.

Table 7-7 shows the PSHA/DSHA comparison for the one-second acceleration and the PSHA/DSHA ratio at each of the selected sites. These ratios are shown graphically in Figure 7-2.

Based on this comparison, the following recommendations were proposed:

1. Adopt the 5% in 50 years hazard level for development of a design spectrum.
2. Ensure sufficient conservatism (1.5 safety factor) for minimum seat width requirement. This conservatism is needed to enable to use the reserve capacity of hinging mechanism of the bridge system. This conservatism shall be embedded in the specifications to address unseating vulnerability. It is recommended to embed this safety factor for sites outside of California.
3. Partition Seismic Design Categories (SDCs) into four categories and proceed with the development of analytical bounds using the 5% in 50 years Hazard level.

Table 7-4. Probabilistic Spectral Acceleration for 10% and 5% in 50 Years

CITY	10%/50 yr Ss, g	10%/50 yr S1, g	5%/50 yr Ss, g	5%/50 yr S1, g
Daly City	1.60	0.78	2.15	1.12
San Francisco	1.15	0.53	1.45	0.69
SFOBB	1.26	0.50	1.57	0.62
Berkeley	1.65	0.63	2.19	0.83
Benicia Martinez	1.24	0.43	1.58	0.55
Los Angeles	1.20	0.41	1.60	0.54
Vincent Thomas	1.02	0.37	1.47	0.56
Long Beach	0.96	0.35	1.30	0.49
Coronado Bridge	0.60	0.22	0.89	0.34
Seattle	0.73	0.24	0.99	0.33
Tacoma North	0.68	0.23	0.89	0.30
Salt Lake City	0.69	0.24	1.10	0.42
Salt Lake City	0.68	0.24	1.09	0.42
Evansville	0.25	0.07	0.40	0.11
St. Louis	0.23	0.06	0.36	0.10
Paducah	0.54	0.11	0.97	0.24
Union City	0.53	0.12	1.06	0.27
Memphis	0.38	0.09	0.75	0.19
Memphis	0.40	0.09	0.80	0.20
Charleston	0.31	0.06	0.69	0.15
Phoenix	0.09	0.03	0.12	0.04

Table 7-5. Spectral Acceleration (Type B & D Soil) for 5% in 50 Years

CITY	Type B 5%/50 yr Ss, g	Type B 5%/50 yr S1, g	Type D 5%/50 yr S_{DS} , g	Type D 5%, 50 yr S_{D1} ,g
Daly City	2.15	1.12	2.15	1.67
San Francisco	1.45	0.69	1.45	1.04
SFOBB	1.57	0.62	1.57	0.93
Berkeley	2.19	0.83	2.19	1.24
Benicia Martinez	1.58	0.55	1.58	0.83
Los Angeles	1.60	0.54	1.60	0.81
Vincent Thomas	1.47	0.56	1.47	0.84
Long Beach	1.30	0.49	1.30	0.73
Coronado Bridge	0.89	0.34	1.02	0.58
Seattle	0.99	0.33	1.10	0.58
Tacoma North	0.89	0.30	1.01	0.54
Salt Lake City	1.10	0.42	1.17	0.67
Salt Lake City	1.09	0.42	1.15	0.66
Evansville	0.40	0.11	0.60	0.26
St. Louis	0.36	0.10	0.55	0.24
Paducah	0.97	0.24	1.08	0.46
Union City	1.06	0.27	1.15	0.50
Memphis	0.75	0.19	0.89	0.39
Memphis	0.80	0.20	0.94	0.41
Charleston	0.69	0.15	0.86	0.34
Phoenix	0.12	0.04	0.19	0.09

Table 7-6. One-Second Spectral Acceleration Comparison to USGS 1996

CITY	10%/50 yr S1, g	5%/50 yr S1, g	10%/50yr 1996 S1,g	5%/50yr 1996 S1,g	Ratio 1	Ratio 2
Daly City	0.78	1.12	1.08	1.50	1.38	1.35
San Francisco	0.53	0.69	0.64	0.83	1.22	1.20
SFOBB	0.50	0.62	0.62	0.79	1.24	1.27
Berkeley	0.63	0.83	0.65	0.86	1.03	1.04
Benicia Martinez	0.43	0.55	0.55	0.69	1.27	1.25
Los Angeles	0.41	0.54	0.42	0.54	1.02	1.00
Vincent Thomas	0.37	0.56	0.41	0.58	1.10	1.03
Long Beach	0.35	0.49	0.42	0.60	1.20	1.23
Coronado Bridge	0.22	0.34	0.21	0.31	0.95	0.92
Seattle	0.24	0.33	0.22	0.32	0.92	0.97
Tacoma North	0.23	0.30	0.20	0.28	0.88	0.93
Salt Lake City	0.24	0.42	0.21	0.42	0.86	1.00
Salt Lake City	0.24	0.42	0.21	0.40	0.87	0.96
Evansville	0.07	0.11	0.06	0.12	0.92	1.08
St. Louis	0.06	0.10	0.05	0.10	0.85	0.99
Paducah	0.11	0.24	0.09	0.20	0.81	0.83
Union City	0.12	0.27	0.09	0.21	0.78	0.79
Memphis	0.09	0.19	0.07	0.16	0.81	0.84
Memphis	0.09	0.20	0.07	0.17	0.78	0.84
Charleston	0.06	0.15	0.07	0.17	1.17	1.11
Phoenix	0.03	0.04	0.03	0.04	1.11	1.02

Table 7-7. Probabilistic to Deterministic Comparison of One-Second Acceleration

CITY	Det S1, g	5%/50 yr S1, g	Ratio
Daly City	0.85	1.12	1.31
San Francisco	0.44	0.69	1.56
SFOBB	0.30	0.62	2.07
Berkeley	0.49	0.83	1.67
Benicia Martinez	0.31	0.55	1.76
Los Angeles	0.57	0.54	0.94
Vincent Thomas	0.63	0.56	0.89
Long Beach	0.51	0.49	0.95
Coronado Bridge	0.47	0.34	0.72
Seattle	0.48	0.33	0.68
Tacoma North	0.18	0.30	1.63
Salt Lake City	0.53	0.42	0.79
Salt Lake City	0.53	0.42	0.79
Evansville	0.09	0.11	1.24
St. Louis	0.08	0.10	1.28
Paducah	0.24	0.24	1.00
Union City	0.23	0.27	1.13
Memphis	0.17	0.19	1.13
Memphis	0.18	0.20	1.12

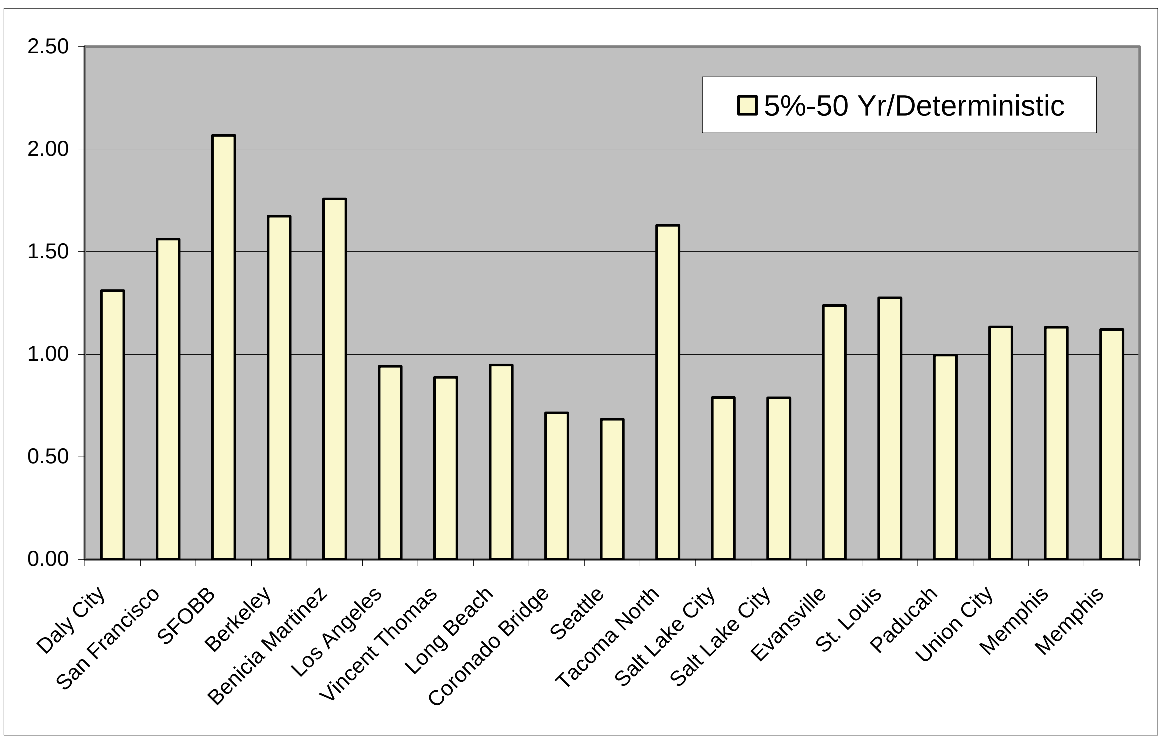

Figure 7-2. Probabilistic to Deterministic Ratio at Selected Sites

Technical Assistance Agreement Between AASHTO and USGS

Following the recommendations of the Task 6 Report discussed above, the AASHTO T-3 committee entered into an agreement with USGS to prepare two types of products for use by AASHTO. The first product was a set of paper maps of selected seismic design parameters for a 7% probability of exceedance in 75 years (equivalent to 5% in 50 Years recommended by the Task 6 Report). The second product was a ground motion software tool to simplify determination of the seismic design parameters.

The LRFD Guide Specifications use spectral response acceleration with a 7% probability of exceedance in 75 years as the basis of the seismic design requirements. As part of the National Earthquake Hazards Reduction Program, the U.S. Geological Survey's National Seismic Hazards Mapping Project prepares seismic hazard maps of different ground motion parameters with different probabilities of exceedance. However maps were not prepared for the probability level required for use by these guidelines. These maps were prepared by the U.S. Geological Survey under a separate Technical Assistance Agreement with the American Association of State Highway and Transportation Officials (AASHTO), Inc. for use by AASHTO and in particular the Highway Subcommittee on Bridges and Structures.

The set of paper maps covered the fifty states of the U.S. and Puerto Rico. Some regional maps were also included in order to improve resolution of contours. Maps of the conterminous 48 states were based on USGS data used to prepare maps for a 2002 update. Alaska was based on USGS data used to prepare a map for a 2006 update. Hawaii was based on USGS data used to prepare 1998 maps. Puerto Rico was based on USGS data used to prepare 2003 maps.

The maps included in the map package were prepared in consultation with the Subcommittee on Bridges and Structures. The package included a series of maps that provide:

- The peak horizontal ground acceleration coefficient, PGA
- A short period (0.2 sec) value of spectral acceleration coefficient, Ss
- A longer period (1.0 sec) value of spectral acceleration coefficient, S1

The maps are for spectral accelerations for a reference Site Class B.

The ground motion software tool was packaged on a CD-ROM for installation on a PC using a Windows-based operating system. The software includes features allowing the user to calculate the mapped spectral response accelerations as described below:

- PGA, Ss, and S1: Determination of the parameters PGA, Ss, and S1 by latitude-longitude or zip code from the USGS data.

- Design values of PGA, Ss, and S1: Modification of PGA, Ss, and S1 by the site factors to obtain design values. These are calculated using the mapped parameters and the site coefficients for a specified site class.

In addition to calculation of the basic parameters, the CD allows the user to obtain the following additional information for a specified site:

- Calculation of a response spectrum: The user can calculate response spectra for spectral response accelerations and spectral displacements using design values of PGA, Ss, and S1. In addition to the numerical data the tools include graphic displays of the data. Both graphics and data can be saved to files.
- Maps: The CD also include the 7% in 75 year maps in PDF format. A map viewer is included that allows the user to click on a map name from a list and display the map.

Conclusion

A new, long awaited, AASHTO LRFD Guide Specification for Seismic Design of Highway Bridges has been developed under the guidance and support of the AASHTO T-3 Committee on Seismic Design and several participating member and volunteer states. The Guide Specification and new seismic hazard for a 1000 year return period were adopted unanimously by AASHTO at the yearly meeting on July 12, 2007.

The Guide Specifications were developed specifically for ordinary, girder and slab type bridges, using a Displacement Based Approach with design factors calibrated to prevent collapse and reflect both the inherent reserve capacity to deform under imposed seismic loads and to accommodate relative displacements at the supports and articulated connections. A new seismic hazard map was developed by USGS for 7% probability of exceedance in 75 years (i.e. approximately a 1000 year return period) specifically for AASHTO and is now under the control of AASHTO. The selected hazard for design was made considering large historical earthquakes, without compromising collapse prevention, using calibrated design factors. Uniform hazard design spectra are constructed using the Three Point Method with the new AASHTO/USGS Maps for the PGA, 0.2 second, and 1.0 second horizontal ground accelerations and corresponding HEHRP Site Class Spectral Acceleration Coefficients.

As seen from the approach taken to address the appropriate hazard level for the new LRFD Guidelines, the performance criteria associated with typical bridges needed to be considered in order to select an appropriate return period that meets the objective of the stakeholders recognizing that the LRFD Guidelines are minimum requirements that can be upgraded regionally or on specific essential or critical bridges.

Improvement of Bridge Safety Through Forensic Studies

8

George C. Lee and Jerome S. O'Connor

The objective of this paper is to build a case for establishing a comprehensive bridge failure database in one repository and using it to improve the reliability of our nation's bridges. If detailed information about each catastrophic event is made available in a usable form, researchers can reconstruct the event digitally and conduct an in-depth analysis of the capacity of the structure under various circumstances so that future designs can be improved. Additionally, computer simulations can help to get a better understanding of the hazard loading effect on bridges so that a uniform platform of hazard evaluation and comparison can be established based on bridge damage due to various hazards.

In the following paragraphs, a plan is outlined establishing a systematic approach to accident data collection and subsequent analyses.

For the purposes of this paper, the following definitions are employed.

- *Failure:* a structure is no longer able to function as originally intended, i.e., a measure of functionality.
  - *Structural Failure:* a structural element is no longer able to function as originally intended because its capacity has been exceeded or it has been displaced beyond acceptable limits.
  - *Collapse:* the loss of a bridge span due to gravity.
  - *Partial Collapse:* the loss of one or more primary structural members of a span have undergone severe deformation or displacement such that safe passage is no longer possible.
- *Damage:* the physical change of a structural element for the worse, i.e., a description of structural condition.
  - *Severe Damage:* the physical change of a structural element that prevents it from functioning as designed. (*Light* or *Moderate* can be used to identify lesser degrees of damage)
- *Fail-*Safe *Design*: A design philosophy that insures that the failure of the system does not result in an unsafe condition. For instance, provision of multiple load paths allows one to pick up the load in the event that a structural member in another fails. In bridge design, structural redundancy, internal redundancy, span continuity, wide bridge seats, and similar measures are ways of contributing to the inherent safety of a bridge.

Factors that Jeopardize Bridge Safety

Keeping a bridge in good shape is the first line of defense against normal hazard events. Condition of the bridge should be monitored and maintained so that it performs as intended by design. However, there is still some level of risk associated with operating a bridge, because unexpected occurrences of extreme events can be expected. We just do not know what will happen, when it will occur or which of our bridges will be affected.

Bridge owners can build safer bridges by funding continued research that will result in improved bridge design specifications. Each generation of Highway Bridge can, and should, be better than the previous one. While we refine our approach to designing bridges, we need to remember that we have raised the standard. In certain situations, the difference is severe enough that the older type may need to be retrofitted to remedy a serious design flaw. In other cases, the bridge may be sufficiently safe (i.e., low enough risk), that we continue to use it, and accept the fact that it is not *as* safe as a new bridge. In all cases, we need to establish reasonable criteria and approaches to deal with rare occurrences of large intensity hazards to design economically acceptable bridges.

Dealing with Existing Bridges

It must be acknowledged that as the state-of-the-art moves forward, there are over 585,000 existing bridges (in the United States) that are essentially frozen in time. Unless they receive a retrofit, which is often costly, their ability to resist loads remains unchanged. In cases where the physical condition deteriorates because of environmental or other factors, the structure's integrity actually decreases over time. These older bridges:

- Were often designed for a lower live load carrying capacity that is used today,
- Were not designed to handle lateral loads induced by some hazards such as earthquakes,
- Have details for welding that were designed without benefit of a full understanding of steel fatigue,
- Were built when the strength of materials was not as controlled and predictable as it is today,
- Quality of workmanship may not have been well monitored (e.g., field welding),
- Occasionally have weakened section properties because of inadequate maintenance and/or corrosion,
- Are considered historic and protected from major alteration, even if it would improve performance,
- Cannot realistically be rehabilitated without entirely rebuilding it, or

- Still function under normal loads but cannot be expected to survive extreme loads as well as a new bridge,

Potential Failure Modes

There are numerous scenarios that could compromise the integrity of a bridge and its ability to carry traffic. Below is a partial list of potential modes of failure:

1. Dropped span due to loss of support at a pier due to:
    a. Undermining of the foundation from scour
    b. Loss of a pier due to vehicular or vessel collision
    c. Plastic hinging from earthquake loading
    d. Inadequate seat width
    e. Toppled bearing
2. Severe deformation of one or more primary members due to:
    a. Impact from vehicular or vessel collision
    b. Fire underneath (usually resulting from spilled fuel from a vehicular collision)
    c. Earthquake
3. Excessive tipping of an abutment or pier due to:
    a. Scour
    b. Liquefaction
    c. Vessel collision
4. Excessive lateral displacement such that the structure is unusable. This might be due to:
    a. Wave force and/or flooding
    b. Earthquake loading
5. Structural failure of a member due to:
    a. Inadequate design for today's loads
    b. Brittle fracture resulting from fatigue
    c. Ductile fracture from overload
    d. Stress corrosion
    e. Localized deterioration
    f. Excessively localized loading
    g. Lack of freedom of movement due to corrosion
    h. Unintentional displacement of a member (e.g., a pin in a pin and hanger assembly)
    i. Loss of prestressing in a P/S beam after being subjected to uplift due to buoyancy forces

History has shown that when a complete bridge failure occurs, there is usually one dominant hazard that promulgated the collapse of a bridge, but there is usually more than one contributing factor.

A bridge failure may occur with the occurrence of more than one load effect at the same time, one followed closely by another (cascading events), or one that applies load to a bridge before some deteriorated condition or previous damage has been corrected.

Age

In the case of fatigue and corrosion, damage can occur over time. Even as bridge owners go to great pains to slow deterioration from causes such as these, it is widely accepted as inevitable. These issues are a concern because they slowly compromise the integrity of the bridge, but the problem does not become paramount unless a catastrophic failure becomes imminent or actually occurs.

The authors propose that the *rate of failure* is most important. Even a highway agency with tight budget constraints needs to avoid a catastrophic collapse that would endanger lives. Degradation is inherently accepted as long as it is slow enough that action can be taken before an unexpected failure or catastrophe occurs. Many agencies must get as many years as possible out of a bridge until funding is available.

In cases where damage occurs over a period of time, hindsight often tells us that measures could have been taken to insure that the bridge was being taken care of and functioning as designed. Since operational issues such as maintenance and inspection can influence the likelihood of failure, they need to be dealt with. They are one of the few variables that we have some degree of control over. Even though we cannot predict with certainty the magnitude and recurrence of an extreme event that a bridge might be exposed to, a bridge owner can at least mitigate the prospects of damage by exerting regular control over the slow moving causes of deterioration.

Historical Performance

Although not every bridge collapse captures national headlines, unfortunately, they occur on a fairly regular basis. Several organizations, such as the New York State Department of Transportation, already track bridge information about bridge failures (NYSDOT 2006). Jean-Louis Briaud of the Texas A&M University reports that 1,500 bridges collapsed in the U.S. between 1966 and 2005 (ASCE, 2008). From a review of these basic datasets, it appears that in the United States alone, there is an average of about three failures every month. This generates an abundance of important data that is available for analysis.

Bridge Hazard Categories

Bridge hazards can be divided into the following broad categories:

I Natural Hazards
(e.g., flooding, earthquake, hurricane)
II Man-made – Unintentional
(e.g., condition related, accidents such as collisions, learning experiences, deficient designs, lack of maintenance, lack of enforcement)
III Man-made – Intentional
(i.e., fanatical acts of terrorism)

Though incidences such as an overweight truck crossing a bridge posted with a weight restriction could be classified as intentional, most bridge owners would consider these unintentional. Even though driver negligence or ignorance might lead to the collapse of a bridge, causing failure of the bridge was probably not the objective of the driver.

Past Bridge Failures

Table 8-1 presents some of the more notorious bridges failures that have occurred in the United States.

Most of the county's almost 600,000 bridges were designed for a service life of 50 years or more. A quick glance at the bridge age for each of these catastrophes shows that most of the bridges were years away from the end of their service life. Since age is frequently taken to be indicative of condition, this seems to indicate that deteriorated condition is not a dominant cause of failure, as one might expect.

While those in the civil engineering profession know that additional funds are needed to adequately take care of our *aging* bridge population, the data from historical failures seems to indicate that something else is going on. While it is intrinsically obvious that we need to try to keep our bridges in good *condition*, there are other, apparently more dominant, factors contributing to the bridge failures that have occurred. The fact that we continue to see photos of bridge collapses in the newspapers means that we have not yet adequately identified the causes of bridge failure. It is imperative that the causes of failure are understood so patterns can be identified that would be useful in making predictions about which bridges are vulnerable to sudden collapse.

NYSDOT's Bridge Collapse Database, as well as others, shows that more than half of the bridges that fail, do so as a result of hydraulic scour, or undermining, of the foundation (see Figure 2-1 in Chapter 2). For this reason, recent research has focused on developing realistic estimates of scour depth and on developing reliable methods for monitoring scour in the field. Figure 8-1 illustrates such a system.

Table 8-1. Selected Bridge Failures in the United States

Year	Description of Bridge Collapse	Bridge Age (years)	Primary Cause (Hazard)	Lessons Learned
1940	Catastrophic collapse of the **Tacoma Narrows** suspension bridge in Washington State. Video: *http://www.youtube.com/watch?v=3mclp9QmCGs*	< 1	Wind	Importance of harmonics / resonance vibration
1967	Collapse of the 41-year-old **Silver** (suspension) **Bridge** across the Ohio River at Point Pleasant, WV Video: *http://www.wvculture.org/history/av.html*	39	Steel Corrosion & Fatigue	Need for an inspection program
1980	**Sunshine Skyway Bridge** near St. Petersburg, FL, 1261 feet of bridge fell.	10	Vessel Collision	Need to design & armor piers to protect against ship collision
1983	**Mianus River Bridge** on Interstate 95 in Connecticut. A 100-foot span fell.	29	Inspection	Pin & Hanger, Redundancy, Corrosion
1987	Total collapse of Interstate 90 **Schoharie Creek Bridge,** NY Summary Report: *http://www.eng.uab.edu/cee/faculty/ndelatte/case_studies_project/Schoharie.htm*	33	Scour	Need for rigorous inspection and maintenance
1989	**San Francisco-Oakland Bay Bridge** CA, Collapse Video: *http://www.youtube.com/watch?v=AFwJR04qBys* and *http://www.youtube.com/watch?v=7Q0_Z-9839c*	53	Earthquake	Merits of performance based design
2004	Interstate 10 Bridge, **Escambia Bay,** FL, 3300 feet collapsed. Summary Report: *http://www.southalabama.edu/usacterec/ivanimpact.pdf*	~25	Hurricane Ivan	Need to quantify wave force
2007	**I35W in Minneapolis,** MN 13 fatalities Video: *http://www.youtube.com/watch/?v=nerQhIyOwxM*	40	Un-determined	Un-determined

Figure 8-1. Scour Monitoring Device Mounted on a Bridge Pier to Supplement Visual Inspections

Overload, too, is a serious problem that needs to be dealt with. A 2006 audit by the U.S. Inspector General (USDOT) found that inaccurate or outdated maximum weight limit calculations and postings existed. Auditors projected that on over 10.5% of bridges on the National Highway System (NHS), the load rating calculations did not accurately reflect the condition of the structure. Furthermore, 7.8% of bridges were required to have maximum safe weight signs posted on them, but the signs were not posted and overload vehicles were allowed to cross the bridges.

Current Means of Assuring Bridge Safety

The United States established a comprehensive bridge inspection program after the collapse of the Silver Bridge in 1967 (see Table 8-1). This catastrophic event prompted the development and implementation of the *National Bridge Inspection Standards* (NBIS 2007) whereby every bridge carrying public traffic is inspected at least every two years. This program provides the traveling public with some level of assurance that the bridges in the country are safe. The primary purpose of the NBIS is to locate and evaluate any existing bridge deficiencies to assure the safety of the traveling public.

Particular aspects of the NBIS are:

- Visual inspection and qualitative rating of major structural components
- Maintenance of a bridge inventory
- Applicability to any structure on a public road that has a span exceeding twenty feet
- Biennial visual inspections of any existing bridge carrying public traffic
- Underwater inspections at least every five years
- Load rating of bridges
- Posting of weight restrictions
- Annual hands-on inspection of fracture critical structural members
- Use of Quality Control and Quality Assurance practices
- Special and Damage inspections

Other measures routinely used to insure that bridges are safe:

- Continual refinement of bridge design specification by American Association of State highway and Transportation Officials (AASHTO)
- Regular funding of a bridge rehabilitation & replacement program
- Studies to assess hydraulic capacity and scour issues
- Increased frequency of inspection
- Continuous or intermittent monitoring of unique problems (see Figure 8-1)

Indices of Bridge Safety

After the 2007 collapse of the I35-W bridge in Minneapolis, Minnesota, there was much attention given to the fact that the bridge was designated as structurally deficient by the Federal Highway Administration (FHWA). Bridges are classified as "structurally deficient" if they have a general condition rating for the deck, superstructure, substructure or culvert of four (i.e., poor condition) or less, or if the road approaches regularly overtop due to flooding. The classification "Structurally Deficient" is used to determine eligibility for federal bridge replacement and rehabilitation funding.

Data in the National Bridge Inventory (NBI) is used to generate federal sufficiency ratings. Inventory data and condition ratings assigned by bridge inspectors are used to compute a sufficiency rating for each bridge. This number, ranging from 0 to 100, is used to determine a particular bridge's eligibility for Federal funding. The weighted formula considers many factors, giving consideration to a bridge's structural adequacy and safety, its serviceability and functionality, and its essentiality. In addition to the bridge inspector's condition rating for the superstructure and substructure, it uses such factors as the structure type and width, number of lanes, volume of traffic carried, waterway adequacy, detour length and necessity as part of the National Defense Highway Network. The sufficiency rating does not necessarily indicate a bridge's ability to carry traffic loads. It helps determine which bridges may need repair or replacement, but not which bridges are safe and which could collapse.

Although these commonly used terms provide a useful reference for the general condition of a bridge and are useful for allocating funds to states, they were never intended to be an indicator of risk.

Certain agencies, such as New York State Department of Transportation, have initiated a risk-focused program with the objective of prioritizing bridges in a population according to their vulnerability to various hazards. (O'Connor, 2000) Assessment procedures are used to produce a rating that can be used to supplement bridge inspection ratings that are purely condition oriented. The use of both ratings provides a better means of allocating resources for intensified inspections or remedial work.

Need for Improvement

Although the NBIS has increased the safety of our existing highway infrastructure, it has not been sufficient to prevent further bridge collapses and loss of life. One important issue to be dealt with before we can achieve better collapse-resistant bridges is to develop a better understanding of the root causes of failures that have occurred in the past. Though there are historical accounts and a few forensic studies of the worst failures, there is currently not a central repository for the detailed and specific data about the load effects of extreme hazards. Likewise, information about the response of the bridge during an event such as the forces carried by critical structural members at failure, is not routinely available and recorded. At the same time, there are no uniform criteria to define or describe damages of different types of bridges.

NBIS, the hallmark of our nation's plan to assure the safety of the traveling public, has the clear but limited objective of identifying deficiencies in existing bridges. These standards are not intended to be used to glean information from the historical performance of bridges for improvement of bridge design or retrofit practice. The core of the program is intermittent visual condition inspections that provide a qualitative assessment of the extent of damage.

Table 8-1 presents some of the more notorious bridge failures that have occurred in the United States. This sampling of historical events gives an indication of the severity of the problem. To varying degrees, these incidences led to loss of life, direct financial expense, and negative impact on the regional economy due to the loss of a critical link in the transportation network. It also serves to undermine the confidence of the traveling public. The public has a right to expect that fail-safe bridges will be provided in exchange for their continued investment in our nation's infrastructure.

The relationship between bridge age and bridge failure is but one connection that can be drawn. With closer scrutiny of additional data (such as the member sizes and their condition, meteorological conditions at the time of failure, live load conditions, and other circumstances), additional correlations may become known that are not as readily apparent. The availability of advanced analytical and modeling software, now makes it possible for engineers to recreate the incident to a degree of preciseness that was not available in then past. If the medical field can create 3D images of the human body to diagnose and treat patients, should not engineers also fully tap the power of technology in the performance of their jobs? Lives are at stake in both instances.

The *AASHTO LRFD Bridge Design Specifications* (AASHTO 2002) are used for the design of almost all new highway bridges. It is a logical approach, based on probabilities, to ensure constructability, safety, serviceability, inspectability, economy, and aesthetics. The extreme event load in the specifications, however, could benefit from additional calibration. To address newly defined hazards, and to identify and respond to these challenges in bridge design, understanding hazards and their potential destructive impacts requires careful analyses and scrutiny of past events and failure cases.

Value of Forensic Studies

Knowing the condition of a structure would go a long way toward keeping it operable instead of being added to the list of failed structures. What you measure, you can improve. In the words of Lord Kelvin, devisor of the Kelvin temperature scale: "To measure is to know" and "If you cannot measure it, you cannot improve it."

Since it is difficult, if not impossible, to anticipate all collapse scenarios that might occur over the service life of a bridge, it is logical to draw insight from historical performance of the bridge population as a whole. A better understanding of past failure mechanisms can contribute directly to the development of an improved bridge design specification by comparing the extreme hazard load effects on bridges and structural resistance, and improving the extreme event loading calibration. Code developers currently face a dilemma of protecting a bridge from multiple hazards simultaneously such that the hazards are considered equitably yet that the design optimized to safeguard life, minimize damage, and be cost effective. Similar difficulties exist with regard to the management of existing bridges.

A cursory review of just a few bridge failures is able to hint at the potential benefits of tracking forensic data more closely and making it available to researchers. Questions that could be addressed in a study of a more complete database are:

1. On what topics should scarce research dollars be expended?
2. Is there any correlation between a structure's age and potential for failure? Is there a correlation related to structure type, design code used, material type, etc.?
3. Is adequate attention being dedicated to the hazards and modes of failure that are most frequent?
4. Would additional resources committed to more intensive bridge inspections yield a significant benefit?
5. Would additional maintenance activity result in fewer collapses or longer service life?
6. Is a bridge that has been carefully designed and maintained for a long service life safer than other bridges?
7. Can new indices be developed that do a good job of prioritizing risk (i.e., assessing safety)?
8. Is current design practice adequate for extreme hazards?
9. Can current design practice be improved to better handle multiple hazards?
10. Are there cases where multiple hazards happened simultaneously? If so, which hazard could be considered dominant?
11. How can the load effects on bridges be compared for different hazards?
12. How can the uncertainty of the hazard event be assessed?

13. What can be done to reduce the occurrence of accidental damage to bridges?
14. What loads are appropriate when designing against extreme events?
15. What can be done to cost-effectively enhance the safety of existing bridges?
16. Are there better inspection methods (e.g., NDE methods) that should be used during NBIS inspections?
17. Are there programmatic changes that would be beneficial? That is:
    a. Are adequate resources being dedicated to bridge inspection?
    b. Are more frequent inspections warranted for certain types of structures?
    c. Should specialized training be required to be provided to bridge inspectors?
    d. Should all bridge inspectors be certified professional engineers?
    e. Is an element specific inspection and rating better than the current approach?
    f. Is current data collection and reporting adequate?
    g. Is good access readily available to bridge inspectors?
    h. Is the collected data being shared among bridge owners so they can benefit from the experience of others?

The collection and cataloguing of failure and performance data also provides a bridge management benefit. With sufficient data, the performance of particular bridge elements or a structure as a whole can be plotted versus time. These deterioration curves can be used as a tool for allocation of maintenance resources, to program bridge replacements, or used in conjunction with a structural health monitoring protocol to keep close tabs on a significant bridge so that unexpected changes in condition can be anticipated or immediately detected.

Proposed Relational Database

Recently, legislators took action to establish the Long Term Bridge Performance Monitoring Program (LTBPP) under SAFE-TEA Lu legislation (FHWA 2005). The LTBPP will provide for the collection of detailed information data on representative in-service bridges that can be used to improve current bridge design and management. The authors of this paper propose a similar proposal but one that is limited to the collection and analysis of data on bridge failures. Much the way an autopsy can help determine the cause of death or a traffic investigation can help determine responsibility for an accident, a thorough analysis after a bridge collapse can contribute to the betterment of bridges in general. Although this type of investigation is already done in some cases, each study is independent the others and does not benefit from any consistent data structure or procedural guidelines. The data and results are also not archived with the intent of building upon the knowledge gained from that one incident or from the collection of investigations as a whole.

National Repository for Forensic Data

Investigative reports prepared immediately after the event such as those prepared by the National Transportation Safety Board (NTSB) are obviously a source of valuable information. However, there are not available in all cases. There is a need for a central repository of information about all bridge failures so the data can be captured and recorded soon after an event when it is freshest and most accurate. In time, personal accounts of what happened change because memories fade and the location of important files are forgotten as office staff changes over the years. If this data can be formally archived for future reference, it will assist investigators striving to eliminate the causes of bridge failure. The entire collection of data can be analyzed to search out commonalities and trends among various bridge collapses.

A few databases have already been created to track bridge failures but none captures sufficient background information and collapse data to simulate the failure event in such detail that recommendations could be made for changes in the design codes.

A comprehensive relational database, as a minimum, preserves accurate information so that it will be available for future generations of bridge designers. It may even suggest the type of additional data that should be collected from in-service bridges so a better understanding can be obtained about imposed loads.

The "forensic file" for each collapsed bridge will contain records on:

- Basic inventory information (general information such as name, features carried and crossed, location, year built, special significance, etc)
- Historical maintenance records
- Hazard characteristics (ground acceleration records, wind velocities, wave heights)
- Structure configuration, materials, and other design information
- Current condition and changes documented in historical inspection reports
- The environment condition (Local soil conditions, river and ship related information)
- Meteorological conditions at time of failure
- Traffic conditions at the time of failure
- Other pertinent information that can be obtained

In addition to tracking actual bridge failures, the same database could be used to enter data for "near misses" or "successes." It does not take a bridge collapse to obtain relevant information about loads and the ability of a structure to resist them. For instance, if a barge hits a bridge pier but does not cause damage, it is still an important data point that can be obtained. This mass and velocity data can be used to calibrate digital bridge simulations. Taking advantage of these unplanned real-world situations can greatly add to the information that is available through experimentation.

Sources of Data

For any given bridge failure, potential sources of data are:

- Record Plans (As-built drawings)
- NBI and owner's supplemental inventory data
- Bridge inspection reports with ratings and comments of visual observations
- In-depth bridge inspection reports
- Section loss or crack measurements
- Other health monitoring data that has been collected
- Progressive scour depth measurements
- Structural Integrity Evaluations
- Historical maintenance records
- Plans from any repairs or retrofit
- Owner's bridge history file
- Load rating calculations
- Bridge vulnerability assessment data
- National Transportation Safety Board (NTSB) investigation reports

Some of this information may not be routinely recorded by each agency.

Information on the hazard will be available at:

- National Weather Service
- NOAA
- USGS
- Local meteorological weather stations

Data Collection

Since detailed information is essential for the quantitative understanding of historic bridge failures, a concerted effort will be needed to assemble as much information as possible. In almost every case, the bridge owner and the state DOT possess the most up-to-date and most accurate information so close communication between them and researchers will be necessary.

Some information should be tracked as basic inventory data that is not recorded at the present time. With further study about the relationship between failures, it is possible to present a case for either expanding NBIS or establishing a new, possibly voluntary database that provide this valuable information. Resistance to mandating the collection of new data may occur because of the resources required. The establishment of a database and filling in of data could be done on a voluntary basis by states willing to take a leadership role. Some state DOTs either already collect this information or see the value in establishing a comprehensive database.

International experience may also prove beneficial, especially in countries that employ the AASHTO specifications for design of their bridges (AASHTO 2002). For instance, Taiwan has real-world experience with earthquakes and the related effects such as landslides, flooding, and scour that would provide additional data. Several countries in South America rely on the AASHTO bridge specification for the design of their bridges. Since their bridges are subjected to various hazards, input from those countries would be a source of important data.

Types of Data

Basic inventory information beyond what is required by the national Bridge Inventory (NBI) is necessary to classify bridges and discover linkages among various historical collapses and potential future collapse. For instance, tracking and comparing the primary structural material used for construction might provide a basis for the theory that one type of a bridge is more susceptible to overload collapse than another bridge. Tracking the number of spans and the presence of structural continuity would point out a trend that multiple simple spans are more susceptible to collapse than a multiple span bridge that has continuity over its piers. Is there a trend that non-redundant structure types fail more often than redundant ones? Is it possible that lack of maintenance is a stronger influence on the potential for collapse?

Some highly significant bridge collapses may garner attention when in fact the cause may be from rare set of circumstances that is not likely to be repeated. If the most pertinent data about the overall bridge population and collapse data are collected from as many sources as possible around the world, it will be easier to find correlation between factors that would help determine the potential for failure in the future.

Examples of useful data for assessing the risk of bridge collapse due to any given hazard are:

- Structure type and material (not only steel vs. concrete but type of steel)
- Element level condition ratings
- Design criteria used (i.e., was it designed and built before fatigue was fully understood?)
- Design detailing (especially the presence of details that are known to be undesirable)
- Proximity to traffic lanes or navigable waterways

Case Studies

Detailed information from a national bridge collapse database could be used to digitally reconstruct one or more of the events using finite element analysis. This "accident reconstruction" would involve identification of the hazard, its load effects and response characteristics of the "as-is" structure. The as-is condition of the structure is the bridge as designed with consideration of modifications that occurred during or subsequent to original construction. These variations from the original design may or may not be noted in the record plans or work history file. The as-is condition also takes into consideration any changes that have occurred over time due to repair, rehabilitation, or deterioration that may have occurred due to corrosion or fatigue.

Research in this area can have the greatest impact by addressing situations that immediately relevant to bridge owners. For instance, since steel is the preferred structural material in some states, a "typical" steel bridge can be modeled and evaluated for performance in an earthquake or in a collision situation. If a state DOT traditionally builds multiple span bridges without continuity and without integral joints, the performance of that type of bridge can be assessed to insure that the behavior of the dominant structure type in that area is known.

Implementing the Plan

- Establish organizational responsibility and funding for a central repository for detailed information about bridge collapses. This could be done as a small pooled fund project.
- Identify state DOT's that are willing to collaborate on the establishment of a failure database.
- Create the structure and data definitions for a relational database to store relevant data.
- Design an easy-to-use, web based data input tool.
- Collect data to fill out the database. Make data available to all interested state DOT's.
- Conduct studies to determine relationships between loadings and structural response by creating finite element models for select bridge failures.
- Share the results with bridge owners through AASHTO Technical Committee 5, Loads

Conclusion

A comprehensive compilation of data concerning past bridge failures can shed light on how to design new bridges to lessen the risk of collapses in the future. It can also be used to more effectively inspect, monitor and retrofit existing bridges.

There is currently no national database of bridge failure information. The central repository of bridge failure information outlined in this paper can be used to create a voluntary but nationally accepted set of procedures for collecting and sharing this important data so that the resultant analyses and lessons learned can be easily and effectively utilized by bridge designers and policy makers to improve bridge safety.

Finally, it should be mentioned that, in conjunction with the establishment of a damage/collapse database for highway bridges, a comprehensive structural health monitoring system needs to be in place to collect useful structural response and deterioration data over time and to prevent unexpected collapses. The FHWA Long-Term Bridge Performance (LTBP) Program (Lee and Sternberg, 2008) has initiated a project to periodically monitor a representative sample of bridges nationwide using advanced condition monitoring technologies in addition to visual inspections (*http://www.tfhrc.gov/structure/ltbp.htm*). This LTBP may be the mechanism to develop the proposed national database for bridge forensic studies in the long term.

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Summary

9

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Following the recent occurrence of several disasters worldwide caused by a variety of natural and manmade hazards, a group of researchers at the University at Buffalo initiated a pilot study funded by FHWA to examine the possibility of establishing a research program on selected aspects of the multi-hazard design of highway bridges. This preliminary investigation led to the conclusion that, within the context of LRFD, there is an important need to improve design limit states for combined extreme hazard events. If a more reliable and reasonable set of "demand" characteristics could be established, follow-up studies could then be pursued to improve the "capacity" aspects of LRFD.

After the April 2007 workshop, the editors of this report engaged in further discussions with several of the contributing authors, and shared their views with some individuals and groups interested in and knowledgeable about this subject, including the AASHTO Load and Load combinations Committee (T-5). These discussions, together with the contributions to this report and some further study carried out by the researchers, have clearly reconfirmed that this is an important area of research to pursue and that developing design principles, approaches and guidelines for multiple hazard design of highway bridges is a complex task that requires a large, systematic and well coordinated effort. Based on this conclusion, we have developed a possible roadmap for research and development, which is described in the sections below.

Rationale

Safety is one of the highest priorities among all design considerations of highway bridges. According to national bridge failure records, extreme hazard loadings have caused a significant number of bridge failures nationwide. Between 1969~2006, there were a total of 1,769 failures caused by hazards including earthquake (19), flood (718), vessel collision (36), wind (8) and traffic overload (220). Developing solutions to effectively deal with unexpected hazard loadings has been a significant challenge facing the bridge engineering community.

Under the current AASHTO LRFD specifications, design safety of a highway bridge is measured by a uniform reliability. Within the AASHTO LRFD framework, multiple hazards are considered in such a way that typical highway bridges are designed to resist the potential hazard loadings so that the hazard-related bridge failure rates are comparable to other failures such as fatigue and material deterioration. While efforts to calibrate extreme limit states to match the LRFD intended uniform risk criterion are continuing, critics question their validity and corresponding load factors because there is a lack of sufficient hazard data to accurately predict extreme event intensities and recurrence periods. Indeed, many critical hazard design issues, such as increasing design hazard load vs. reduction of life cycle cost, selection of dominant design hazard vs. resisting all hazards, are complicated for decision makers partly due to the large uncertainties associated with occurrence, magnitude, time, and impact area of these extreme hazard events. Unlike dead and live loads, hazard strikes are rare events, which cannot be predicted as accurately as frequently occurring loads.

In addition, bridge operation, service, maintenance and transportation network planning are concerned about the potential impact of hazard occurrences. In particular, transportation officials and bridge owners need a comprehensive methodology to evaluate and compare different hazards to achieve accurate risk assessment and effective disaster planning.

Scope and Approaches

The objective of the proposed research project is to: (1) identify the critical hazard-related weaknesses and R&D issues in current highway bridge design and practice, and (2) develop a research plan for FHWA to work on these problem areas. Understanding that many problem areas and issues cannot be resolved in a one-size-fits-all approach, this project will seek to use a combination of several tasks and approaches:

- First, as a basic rule-of-fact approach, the project will collect and develop an information repository system with past highway bridge failure data and analysis. The information database will serve as a baseline reference resource for technical and code development bodies to review the experiences and lessons learned from past multiple hazard events.
- Second, to reduce the difficulty of communication due to a lack of sufficient quantitative representation of many multiple hazard-related issues, the project will seek to develop a platform that can categorize relevant issues and key parameters into a simple quantitative format for evaluation, comparison and discussion. This approach will simplify and integrate available knowledge of hazards, probability theories, and fragility and reliability theories to build the comparison and evaluation platform.
- The third component complements the second task by emphasizing calibration issues of extreme hazard events with very limited data.
- The fourth approach will examine problem-specific issues through illustrative examples of current design case studies.
- Finally yet importantly, through a selected panel of experts, a set of problems and issues will be scrutinized periodically with review comments and recommendations for future efforts and possible solutions.

Progress and Deliverables

This project is expected to be carried out in a period of four years, beginning with the publication of this report. The following deliverables are anticipated:

- Report – A Possible R&D Roadmap
- Multiple hazard database development
- Multiple hazard evaluation, comparison and calibration platform
- Multiple hazard bridge design case studies
- Improving multiple hazard bridge design practice – critical issues, possible solutions and future research needs

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Dr. Imbsen has been a pioneer in developing, implementing, teaching and applying seismic design principles to bridges since the San Fernando earthquake in 1971. He was a co-recipient of the AASHTO Dr. L.I. Hewes Award for the development of the first comprehensive seismic design specifications, which remained in effect for fifteen years. He has directed and participated in the seismic upgrading of several major bridges including the Golden Gate Bridge, Benicia-Martinez Bridge, George Washington Bridge, I-40 Mississippi River Bridge in Memphis, Tennessee, Tobin Memorial Bridge.

Dr. Imbsen developed the first FHWA Training Manual for Seismic Design of Highway Bridges in 1976, which has been updated several times. He is the Lead Instructor for the one-week FHWA Workshop designed to educate industry professionals to implement the latest technology for seismic design and retrofitting. Dr. Imbsen has given this workshop over 100 times throughout the United States and in several foreign countries.

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Dr. Kulicki led the design of a new proposed Mississippi River crossing at St. Louis. This design was a cable-stayed design with three planes of cables, two single, leaning pylon towers reaching 435 feet above the roadway, and was approximately 3,150 feet long with a 2,000-foot long main span and width of 222 feet. He served as the Principal-in-Charge of the design team for the award-winning $80 million Second Blue Water Bridge in Port Huron, Michigan. This was the first structure in the United States that was designed using LRFD Specifications with metric units.

He led a 50-member team of experts in the development of the AASHTO LRFD Bridge Design Specifications. Dr. Kulicki was named one of ENR's "Men Who Made Marks" in 1991 and received the George S. Richardson Medal at the 1996 International Bridge conference for that work. He is the author of the AASHTO "Guide Specifications for Load Factor Design of Trusses," for which he was named one of ENR's "Men Who Made Marks" in 1984. In 2002 and 2003 he served as Vice-Chairman of the AASHTO/FHWA Blue Ribbon Panel on Bridge and Tunnel Security. In 2000 he was named “Engineer of the Year” for 2000 by the Central Pennsylvania Engineers Week Council, and received a “Special Citation” from the National Steel Bridge Alliance for contributions to the art and science of bridge engineering in 2001. In 2002 he received a “Life Time Achievement Award” from the American Institute of Steel Construction. Also in 2002 he was named “Engineer of the Year” by the Pennsylvania Society of Professional Engineers. In 2005, he received the “Bridge Design Award” from the Bridge Engineering Association and the Transportation Research Board’s Roy Crum Award, and was elected to the National Academy of Engineering in the Class of 2006.

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Dr. Lee has held leadership positions in numerous professional organizations including: ASCE, Welding Research Council, Structural Stability Research Council, U.S. National Committee on Biomechanics, and Committee on Hazard Mitigation Engineering of the National Research Council. He has served as the editor or as a member of editorial boards of several ASCE and international journals. He is currently the US editor-in-chief of Journal of Earthquake Engineering and Earthquake Vibration. He has received numerous awards and citations including the ASCE Newmark Medal for his research contributions and his leadership role to successfully pioneer the center approach in earthquake engineering research engaging multi-institution, multi-disciplinary team efforts. Dr. Lee received the 2006 Presidential Award for Excellence in Science, Mathematics and Engineering Mentoring.

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Recent assignments have included contributions to the analysis and design of the I-70 cable stayed bridge to be built over the Mississippi River, preliminary design for the St. Croix river crossing, rating analysis of an 1850' span suspension bridge, preliminary designs for a cable stayed bridge over the Monongahela River and independent review analyses of several complex structures.

In addition to performing the initial bridge type study for all cable-stayed alternatives for the I-70 New Mississippi River Bridge, Dr. Murphy developed project-wide site specific seismic design criteria based on expected ground motions from a New Madrid seismic event. He also helped facilitate the extensive public involvement process developed for the I-70 project. Dr. Murphy oversaw the analysis for final design of the I-70 bridge. His contributions included one of the first three-dimensional nonlinear construction stage analyses including time dependent effects ever performed on a bridge of this size, as well as the global traffic and seismic analyses.

For the Monongahela River cable stayed bridge, Dr. Murphy, serving as Project Manager, is assisting in the management, design, and complex analysis for all structural aspects of the project. His responsibilities also include monitoring of scope, schedule, budget, and sub-consultant coordination

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Dr. Yen is FHWA's representative to the National Earthquake Loss Reduction Program and is Chair of the *Sixth National Seismic Conference* steering committee. He also served as past Chair of the technical committees of the Fourth and Fifth National Seismic Conferences. He is the chair of FHWA's National Seismic Engineering Team. He serves as the US-side Chair of the annual US-Japan Bridge Engineering Workshop. He is a member of American Society of Civil Engineering, and a voting member of ASCE's Seismic Rehabilitation of Buildings Committee. He serves as a steering committee member of the Mid-America Seismic Conference for Highways, and is a registered Professional Engineer in the state of Virginia.

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