THE BRIDGE OWNER'S DILEMMA WITH LESS COMMON EVENTS

Susan E. Hida[1]

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 Therefore, the calibration used for the *AASHTO LRFD Bridge Design Specifications* (2007) is no longer applicable because the target reliability, β_T = 3.5, is based on the strength of a SINGLE girder (Nowak, 1999).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

[1] Senior Bridge Engineer, California Department of Transportation, Sacramento

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 1.

Table 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

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.

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

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

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 2.

Table 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 $P(A|B)$. 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 $P(X|Y)$ denote the probability of the earthquake coinciding with the storm in one given year. Then, the annual probability of coinciding events is

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

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

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

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 3.

Table 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 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 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 3 and 4, the distribution of the strongest 100 events in each collapse category for each vulnerability level can be displayed as shown in Table 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 5a, 5b, 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 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 6 generalizes events into 'frequent', 'infrequent', and 'rare' and assumes very conservative annual probabilities of occurrence.

Table 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 7 and 8.

Tables 7a, 7b, 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 8a, 8b, 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 air plane 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.

Acknowledgements

The author thanks Tom Shantz, Caltrans Division of Research and Innovation, for his review of this paper and valuable suggestions. Advice from Prof. Andy Nowak, University of Nebraska-Lincoln, is also appreciated along with comments from Mike Keever, Caltrans Office of Structure Design Support and Earthquake Engineering. The views in this paper do not necessarily reflect those of the California Department of Transportation.

References

American Association of State Highway and Transportation Officials (2007). “AASHTO LRFD Bridge Design Specifications”, 4th Ed. Washington DC.

American Association of State Highway and Transportation Officials (publication pending). “AASHTO LRFD Guide Specifications for Seismic Design of Highway Bridges”, Washington DC.

Ang, H-S. and Tang, W.H. (1975). “Probability Concepts in Engineering Planning and Design”, Volume I—Basic Principles, John Wiley & Sons, New York.

California Department of Transportation, “Amendments to the AASHTO LRFD Bridge Design Specifications”, v03.06.01. Sacramento, California.

California Department of Transportation (2006), “Seismic Design Criteria” Version 1.4. Sacramento, California.

Hida, Susan E. (2007). “Statistical Significance of Less Common Load Combinations”, *Journal of Bridge Engineering*, Vol. 12, No. 3, pp389-393. ASCE, Reston, Virginia.

Ketchum, M., Chang, V. and Shantz, T (2004). *Influence of Design Ground Motion Level on Highway Bridge Costs.* Pacific Earthquake Engineering Research Center Project No. 6D01. Richmond, California

Novak, Andrzej S. (1999). *Calibration of LRFD Bridge Design Code.* NCHRP Report 368, Transportation Research Board of the National Academies, Washington DC.