Seismic performance-based design of bridges with quantitative local performance criteria

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Seismic Performance-Based Design of Bridges with Quantitative Local Performance Criteria

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Abstract: Performance based design of bridges for earthquake resistance is still not explicitly used in design. However, most codes specify a level of performance for bridges under various earthquake inputs. In principle, design rules suggested in the code should meet stipulated performance criteria. However, it has been highlighted in the past that design rules are not directly related with stipulated performance criteria. After the presentation of performance criteria and their relation with post-earthquake functional requirements, the article examines the case of the Canadian Bridge Design Code (CAN/CSA-S6-06). Performance of bridges designed with this code is predicted and compared to expected levels. It is shown that compliance with design rule does not guarantee an adequate performance. This article attempts to correlate qualitative performance criteria with post-earthquake functional requirements, and critically examines code specified design rules and their cost effectiveness. Some implicit design rules are proposed to design new bridges.

1. Introduction

Major seismic events during the past few decades have continued to demonstrate the destructive power of earthquakes, with failures to structures such as bridges, as well as giving rise to great economic losses. Economic losses for bridges very often surpass the cost of damage and should therefore be taken into account in selecting seismic design performance objectives.
The structural engineering community in its transition to performance-based seismic design codes has proposed several methodologies for performance-based seismic design or upgrading. Although it is generally recognized, the significance of economic factors has not been integrated explicitly with the technical issues to develop the methodologies.

Design codes have adopted different approaches to achieve required performance objectives. However, the performance objectives in the design codes are defined qualitatively in terms of design principles called the "seismic design philosophy". It is not clear how design requirements are related to design principles and economic considerations.
To assess code requirements, a vast amount of experimental evidences would be necessary. However, such experiments are normally very expensive. Numerical techniques could be an alternative to these expensive experiments, as they can simulate the experimental behaviour reasonably well when modeled properly (Legeron et al., 2005). A numerical modelling technique is used in this paper after being thoroughly compared with a number of experimental results. With this method, performance of bridges designed by Canadian code (CAN/CSA-S6-06) is compared for the stated performance objectives. Based on these results, some implicit design rules are proposed for the seismic design of bridges.

The article is organized in four parts: (i) definition of performance criteria and relation with post-earthquake functional requirements and methods to predict the performance of bridges; (ii) prediction of performance of bridges designed with the Canadian Code for bridges S6-06; (iii) presentation of a cost-effective way to define seismic design criteria; (iv) presentation of implicit rules of seismic design to comply with cost-effectiveness and post-earthquake functional requirements.

2. Bridge Performance and Post-Earthquake Serviceability

2.1 Performance limit state

Current seismic design codes define different levels of damages depending on the importance of the bridge and the return period of the earthquake event. The performance principles stated in the design codes are just descriptive. Table 1 provides actual performance level that might be related to code based performance principle and are in line with recent development of performance-based seismic assessment (Hose et al., 2000; Lehman et al., 2004). Both qualitative and quantitative performance levels are described in Table 1 and are associated with engineering parameters. Up to the limit state 1A, no damage should take place and expected response is of small displacement amplitude. At this limit state few hairline cracks may be observed. The limit state 1B is considered as the onset of yielding of longitudinal reinforcement. Very minor damage should occur and the bride will remain fully operational after the earthquake. At the limit state 2, spalling of concrete cover may be observed. The deformation of cover concrete is identified to define the limit state. Extreme fibre concrete compression strain of -0.004 is considered to define the limit state, as in ATC-32 (1996). At this limit state, moderate structural damage may take place, bridge may not be fully operational, and only limited service may be allowed for emergency vehicles. At limit state 3, significant structural damage is expected but the bridge should not collapse. The bridge will not be useable after the earthquake and extensive repair may be required. Sometime such repair may not be economically feasible and reconstruction might be necessary. This limit state is best represented by the onset of initial core crushing, onset of bar buckling, or the facture of transverse hoops. Hoshikuma et al. (1997) concluded that crushing of confined concrete and buckling of longitudinal reinforcement may occur when compression stress drops below $0.5f'_{cu}$. Based on the recommendation, onset of initial core crushing has been taken as the point where $\epsilon_c=\epsilon_{cc50}$. Berry and Eberhard (2005) proposed a model to predict the likelihood of the onset of buckling of longitudinal bars in a reinforced concrete column for a given level of lateral deformation. Onset of buckling has been considered as the recommendation of Berry and Eberhard (2005), which takes into account the effective confinement ratio, axial load ratio, aspect ratio, and longitudinal bar diameter. The ultimate limit strain (ϵ_{cu}) is computed with the energy balance method (Mander et al., 1984) in which stress-strain relationship of concrete is integrated numerically and the ultimate limit strain corresponds to the strain where energy absorbed by the confinement reinforcement is equal to the difference of the energy absorbed by the confined concrete and unconfined concrete. The previous description of performance of a bridge can be summarized in a graph showing the performance of a bridge as a function of the ground acceleration. Figure 1 reprents such a graph on which all the functional performance (post-earthquake serviceability) have been indicated.

2.2 Analytical model for seismic performance assessment

An analytical model for seismic performance assessment of bridge pier has been developed in Sheikh et al., 2007. The model forms an analytical tool that reproduces most of the important features of reinforced concrete bridge piers under the action of an earthquake event. The model can well predict the force displacement characteristics of bridge piers considering both flexural and shear behaviour.

Table 1. Performance level and relation with functional post-earthquake requirements

Limit states (LS)	Operational performance level	Post earthquake serviceability	Qualitative performance description	Quantitative performance description	Repair
1A	Fully Operational	Full service	Onset of hairline cracks	Cracks barely visible	No repair
1B			Yielding of longitudinal reinforcement	Crack width <1 mm	Limited epoxy injection
2	Delayed Operational	Limited service	Initiation of inelastic deformation; onset of concrete spalling; development of longitudinal cracks	Crack width: 1-2 mm $\varepsilon_c = -0.004$	Epoxy injection; concrete patching
3	Stability	Closed	Wide crack width/ spalling over full local mechanism regions; buckling of main reinforcement; fracture of transverse hoops; crushing of core concrete; strength degradation	Crack width>2 mm $\varepsilon_c=\varepsilon_{cc50}$ (initial core crushing) $\varepsilon_c=\varepsilon_{cu}$ (fracture of hoops) $\varepsilon_s>0.06$ (longitudinal reinforcement fracture)	Extensive repair / reconstruction
f'_c=axial strain of concrete; ε_{cc50}=post peak axial strain in concrete when capacity drops to 50% of confined strength; ε_{cu}= ultimate strain of concrete; ε_s=tensile strain at fracture					

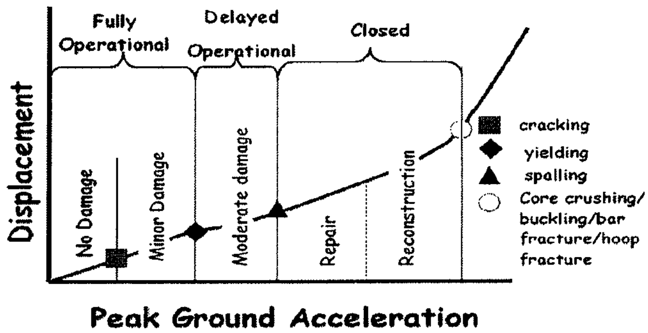

Figure 1: Relation between serviceability of a bridge and performance limit states

To evaluate the capability of the model, the experimental results of 10 columns tested under cyclic loading by Lehman et al. (2004) have been used. Variable of the test covers main parameters of interest for typical bridge piers such as aspect ratio, longitudinal reinforcement ratio, spiral reinforcement ratio, axial load ratio, and length of well confined region adjacent to the zone where plastic hinging is anticipated. Due to the space limitations, analytical predictions of piers 415, 430, and 815 have been reported herein (Figures 2, 3 and 4). It is important to note that the model not only predicts very well the overall behaviour, but all the limit states (LS) as well. For example, in all cases, yielding (LS-1B) is well predicted, as well as initial spalling (LS-2), initial core crushing, buckling of bars and hoop fracture (LS-3) (Fig. 2, 3, 4). The response of the 7 other columns tested by Lehman et al (2004) are also very well predicted globally (overall response) as well as locally (local limit states), as demonstrated in Guiziou (2006). Other predictions performed on shear sensitive columns have demonstrated that the model is effective in predicting shear failure as well (Sheikh, 2007).

3. Performance of Bridge Design with the Canadian Code for Bridges S6-06

Canadian Code for bridges S6-06 has adopted performance requirements as a general principle. Bridges are classified as "lifeline" for critical bridges, "emergency" for the bridges that would need to be used in case of a 475-yr return period earthquake, and "other" bridges being all the ones that can be damaged during an earthquake of 475-yr return period. They are summarized in Table 2 taken in the commentary of the S6-06. Application rules of the code use a constant force-reduction factor R to account for ductility and overstrength, and seismic forces are multiplied by an importance factor I equal to 1 for "other" bridges, 1.5 for "emergency" bridges and 3.0 for "lifeline" bridges. The seismic acceleration is taken as the level corresponding to the 475-year return period earthquake multiplied by the importance factor, *I*.

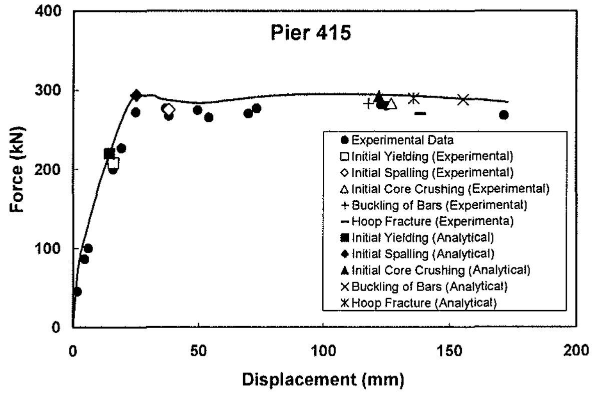

Figure 2: Prediction of performance of Pier 415 tested by Lehman et al 2004

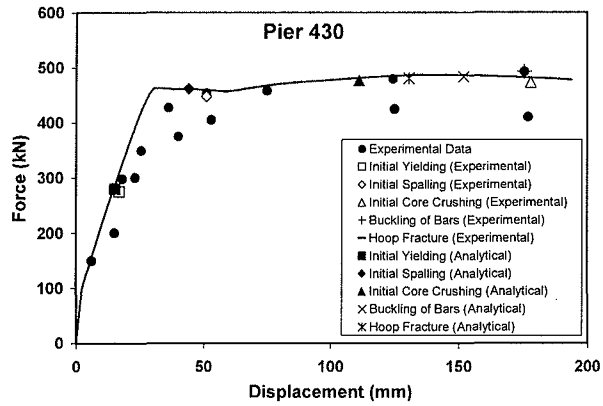

Figure 3: Prediction of performance of Pier 430 tested by Lehman et al 2004

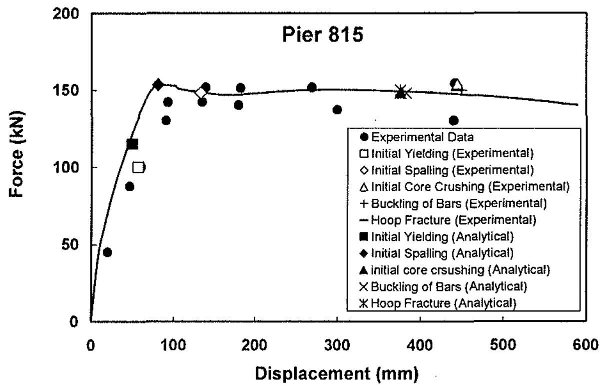

Figure 4: Prediction of performance of Pier 815 tested by Lehman et al 2004

Due to space restriction for this paper, the result of only one four-span bridge with three single circular column 7-m in height is shown here. The bridge has a constant span length of 30 m for a total length of 120m. The design peak ground acceleration is varied from 0.15 to 0.4g, corresponding to a return period of 475 years. The bridge is designed with the appropriate importance factor. As the seismic load is very different from A=0.15 and I=1.0 to A=0.4 and I=3.0, the size of the single column pier had to vary. As the pier is a single column, use of a reduction factor of 3 was used in the design of column as it is stated in S6-06. Varying the size of the pier would lead to a very different period of structures and a variation of seismic loads regardless of A and I. As well, behaviour of single columns can be very different as axial load level decrease with increased diameter of column, and longitudinal reinforcement ratio changes. In order to compare behavior we choose to keep the longitudinal reinforcement ratio constant at about 1%. This would ensure that all piers have a satisfactory behavior under seismic loads and it is also reasonable to assume that the diameter of the column is adjusted according to seismic input. Modelling of the bridge piers has been carried out according to the methodology developed in the previous section. P-Δ effects have also been taken into consideration. The pushover analysis is conducted in order to find out the failure mechanisms and the output is converted into tip displacement (Δ) as a function of peak ground acceleration (*PGA*). Figures 5 to 8 show the results of the calculation for the bridge under consideration with A=0.15 to 0.4 and I=1.0 to 3.0. Figures 5 to 8 show the predicted performance of the 12 bridges used in this study.

It can be observed from the figures that design rules does not always necessarily meet the stipulated seismic performance levels. It is clear from Figure that for low level of earthquake shaking (A=0.15-0.2g), the "other bridge" did not meet the no-collapse performance level, although it has met the repairable performance level in the event of a design earthquake (475-year return period). As well, the performance is oten at the limit of what is accepted by the code in term of performance, and use of higher reduction factors R could result in poor performance of the bridge and probably unacceptable. Notably, the Canandian Code S6-06 specifies a reduction factor R=5 for multibent. In the longitudinal direction, the multibent pier would perform as the single column and the use of R=5 may result in unsatisfactory performance, not only for the 'other bridges' but for 'lifeline bridges' too. Hence, proper evaluation of response modification factor and importance factor is essential and is a subject of further research.

4. Life Cycle Cost Analysis and Optimal Performance

In order to evaluate the optimal economic design of a bridge to resist earthquake, it is interesting to use the concept of life cycle cost (LCC). The LCC is the total cost of a project taking into account cost of damages resulting from earthquake weighted with the probability of occurance of the earthquake. The life cycle cost take into account primary cost (repair or reconstruction cost) as well as secondary cost (economic costs). The complete method to calculate LCC is presented in Sheikh and Legeron 2008.

In order to illustrate the Life cycle cost of a bridge, we consider a simple two-span bridge in the region of Vancouver to demonstrate how to estimate optimal performance based on *LCC*. Peak ground acceleration for 475 year return period earthquake events is 0.3 g in Vancouver. The bridge is considered as an emergency-route bridge and the design life of the bridge is considered as 50 years. It has two spans of 20 m length. The bridge is supported by a single pier of 9 m high and the superstructure unit weight is 150 kN/m. An 11 km detour will be required for 1 km of the roadway in which the bridge is located. The existing facility is posted at 70 km/h and the average speed of the detour is 50 km/h. A constant discount rate of 2 percent is assumed. The bridge is designed for different peak ground acceleration levels. Construction cost of the pier and foundation are calculated based on material cost and labour cost. The construction cost of the superstructure is considered as a constant value of 400,000 CAD.

Seismic damage cost of the bridge has been considered based on the recommendation of HAZUS methodology (NIBS, 1999). Seismic damage cost ratio (damage cost/construction cost) is considered as 0.03 for limited damage (LS-1A), 0.08 for limited damage (LS-1B), 0.25 for moderate damage (LS-2) and 1.0 for extensive damage (LS-3).

Table 2. Functional Performance Requirement in Canadian Code S6-06.

	Bridge		
Return period	Lifeline	Emergency-route	Other
Small to moderate earthquake	All traffic Immediate use	All traffic Immediate use	All traffic Immediate use
Design earthquake (475-year return period)	All traffic Immediate use	Emergency vehicles Immediate use	Repairable damage
Large earthquake (1000-year return period)	Emergency vehicles Immediate use	Repairable damage	No collapse

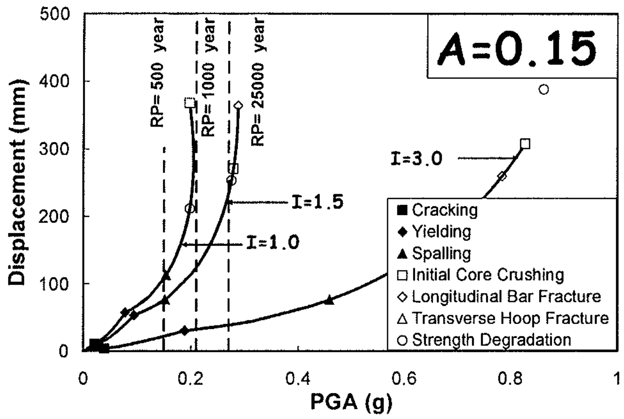

Figure 5: Prediction of performance of bridge designed with A=0.15

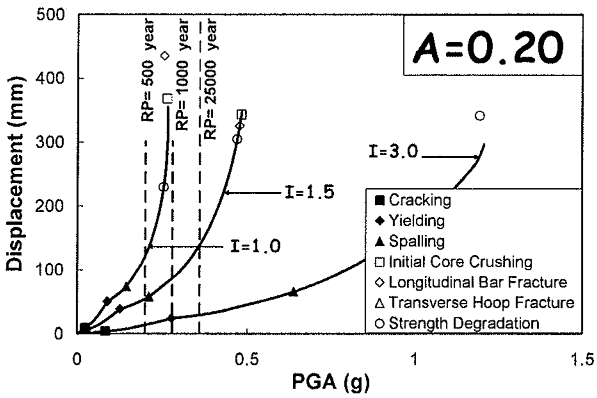

Figure 6: Prediction of performance of bridge designed with A=0.20

User cost can be defined as the sum of the time value cost and vehicle operating cost. Road user costs are calculated according to the procedure developed by New Jersey department of Transportation (2001). The time value cost is considered as 12 CAD/vehicle-hr for car and 21 CAD/vehicle-hr for truck, and vehicle operating costs for car and truck are 0.25 CAD/vehicle-km and 0.45 CAD/vehicle-km, respectively. The restoration period are assumed as 2 days when the limit state 1A is exceeded, 2 weeks when limit state 1B is exceeded, 1 month when limit state 2 is exceeded, and 6 months when limit state 3 is exceeded.

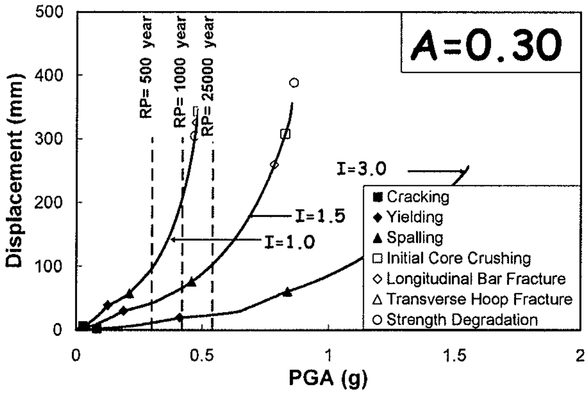

Figure 7: Prediction of performance of bridge designed with A=0.30

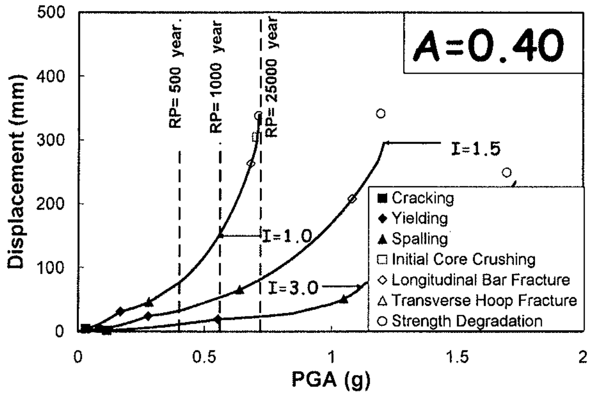

Figure 8: Prediction of performance of bridge designed with A=0.40

For a wide range of intensities, the seismic hazard curve can be approximated as a linear function on a log-log scale (Cornell, 2002). Seismic hazard (annual probability) for each damage state has been calculated by linear interpolation on log-log scale between the two segments of uniform hazard curve.

Life cycle cost has been calculated based on the methodology described earlier in Sheikh and Legeron 2008. Initial construction cost, damage cost and user cost have been calculated as described above. Three cases have been considered based on the average daily traffic using the road. First the bridge is considered to be in a busy roadway considering average daily traffic of 20,000; the second bridge is considered to be in a moderately busy roadway considering average daily traffic of 5,000, and the third bridge is considered to be in a small town with an average daily traffic of 500. It is interesting to note that design earthquake acceleration have only a minor effect on initial construction cost and damage cost. This is reasonable as the superstructure cost remains constant with the increase of design earthquake ground motion, as it is assumed that the bridge pier is the sole structural element designed to withstand earthquake induced ground displacement. The design ground motion has significant impact on the size of the pier and its reinforcement ratio (longitudinal and transverse). However, typically the substructure cost of a bridge (pier and foundation) consists of around 30% of the total construction cost of the bridge.

It is evident for a bridge located in a busy roadway, life cycle cost decreases when the bridge pier is designed for that higher acceleration level (Figure 9). For a moderately busy roadway, life cycle cost slightly decreases with the designed earthquake acceleration level and reach a minimum for return period of earthquake with *PGA* around 0.4g (consistent with importance factor of 1.5) (Figure 10). Whereas in a remote place, life cycle cost is minimum at around 0.3g, which corresponds to 475-year return period earthquake event (Figure 11). It is important to note that construction cost does not change significantly with the design earthquake acceleration level and that user cost is preponderant in the calculation.

Hence, it is economic to design bridge piers for higher earthquake acceleration level when the bridge is located on a busy roadway. This conclusion is based on the result of a simplified bridge model, although it is expected that similar findings may also be observed for real bridges.

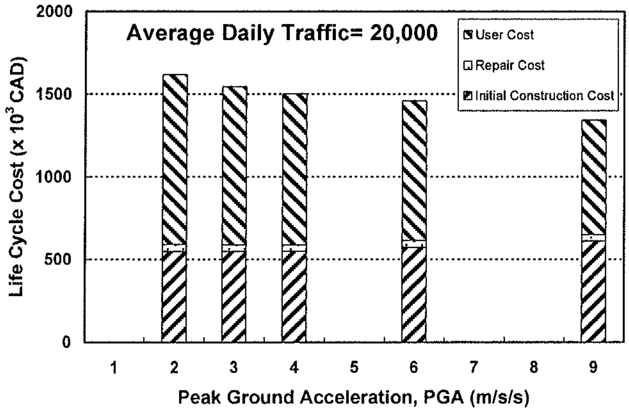

Figure 9: Life cycle cost analysis for bridge with daily traffic of 20,000 vehicles per day

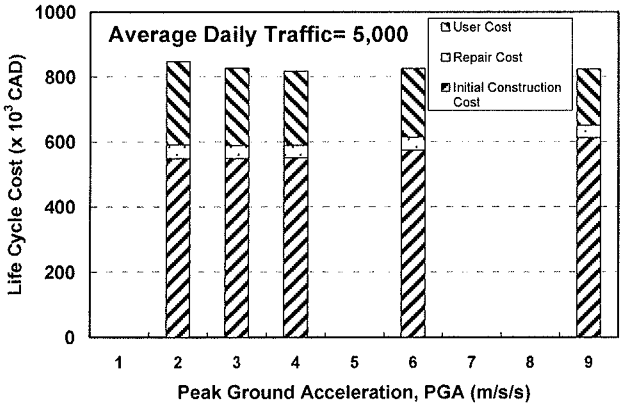

Figure 10: Life cycle cost analysis for bridge with daily traffic of 5,000 vehicles per day

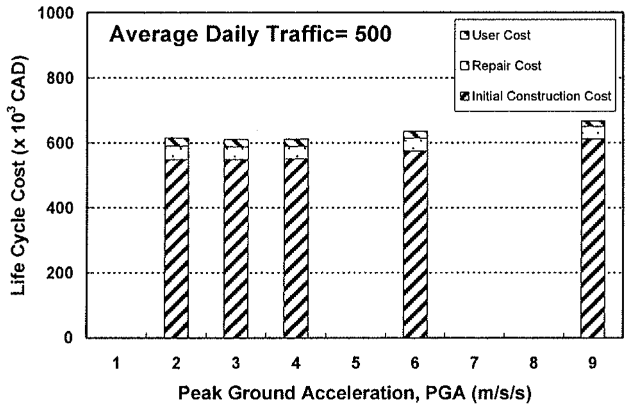

Figure 11: Life cycle cost analysis for bridge with daily traffic of 500 vehicles per day

5. Implicit Design Rules for Performance-Based Design of Bridges

Specifying performance in a design code is very satisfactory on an intellectual and engineering level as it is an assurance of a proper performance at an optimal cost. However, this would suggest an itterative procedure where reinforcement is assumed and checked for performance and if performance is not satisfactory, design is changed accordingly. Performance calculation requires a complex method that is not available to most bridge designers. Specification of performance requirements for large, complex and very important bridges is necessary; large projects designed in North America in the past few years have proposed project specific performance criteria. For most standard bridges, important or not, the use of explicit performance criteria and itterative design procedure is not a viable option. Most design offices are still strugling with current design rules, so it is questionable how they would react to more complexe rule. For the time being, the use of a reduction factor (R-factor) that takes into account the behavior of the structure combined with an importance factor (I) is very interesting for design of standard bridges as it enable a fast and efficient design. Even for complex and very important structures this could provide an initial design that could be checked and modified after proper calculation of performance hence reducing the lengthy itteration process.

Based on the work presented in previous section, the I-factor should be related to the importance of a bridge. The importance of a bridge is also related to its traffic, so bridge with a daily traffic of 5,000 vehicles per day should at least be considered as emergency and bridges with daily traffic of more than 20,000 vehicles per day should at least be designed as lifeline bridges. The basis of design is to assume repairable damages for 475-yr, 1000-yr and 2500-yr return period for "other", "emergency" and "lifeline" bridge respectively seem reasonable. To go from 475 to 1000 and to 2500 years, it is assumed that a I factor of 1.0, 1.5 and 2.0 is used respectively. Alternatuvely, it can be adjusted for the site. Most of the site provides smaller values specifically for the 2500-yr return period. As well, it is possible to propose R-factors for different category of importance of bridges. This is summarized in Table 3 related to expected performance as per the calculation presented in the previous section. With I factor used, it is obvious that the design could be reduced to single level approach with R=3.0, 1.5 and 1.0 for "other", "emergency" and "lifeline" bridge respectively for the 475-yr return period event. The definition of a hazard through an importance factor and design method (in this case using reduction factor accounting for ductility) is interesting and should be kept though, because the design method used could be different and it would therefore avoid any confusion. For exemple, when time history analysis is used, non linear method or when base isolation device is used, this would be a very good way to provide for a homogenuous result.

6. Conclusions

A performance based approach is proposed for optimal seismic design of bridges considering life cycle cost, based on performance limit states that can be related directly to the post-earthquake functionality and repair cost. The methodology could be used for design of a new bridge or retrofitting of an existing one. However, in the methodology, cost of death and injury is not included as such data is scarce. Maintenance cost is also not included as the design earthquake event has insignificant influence on maintenance cost.

This study resulted in the proposal of an implicit approach to design bridges for seismic loads that provide an adequate level of safety and post earthquake serviceability included in a life cycle analysis waranting an optimal economic approach.

More research is necessary to confirm these results for other types of bridges and a more refined analysis accounting for seismic input, dividing in zone of high, moderate and low seismicity, type of seismicity (typicall earsten and western for North America) and soil conditions. This is the objective of ongoing collaborative research between University of Sherbrooke and University of Wollongong.

Acknowledgements

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Table 3: R-factor for the three categories of bridges

Type of bridge	R for 475-yr return period (I=1.0)	R for 1000-yr return period (I=1.5)	R for 2500-yr return period (I=2.0)
Other bridge	3.0 (extensive damage)	4.5 (no collapse)	NA
Emergency bridge	1.5 (full traffic)	3.0 (extensive damage)	4.5
Lifeline bridge	1.0 (elastic)	1.5 (full traffic)	2.0 (limited traffic)

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