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Optimal target performance for cost-effective seismic design of bridges

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OPTIMAL TARGET PERFORMANCE FOR COST- EFFECTIVE SEISMIC DESIGN OF BRIDGES

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ABSTRACT :

A systematic approach is proposed for evaluating the cost-effectiveness of existing bridge design codes based on expected lifecycle cost. In the life cycle cost formulation, costs of construction, damage cost, road user cost, as well as discount cost over the design life of the bridge are considered. The optimal performance is selected on the basis of minimum life cycle cost. The performance of a typical two-span bridge designed according to a current code provision for different earthquake ground motion levels is predicted and optimal target performance is selected based on life cycle cost with different assumptions of user cost. It is demonstrated that life cycle cost should be considered in the design phase of new or retrofitted structures and the target performance significantly depends on the expected average daily traffic using the road.

KEYWORDS: Optimal design, life cycle cost, cost-effective, seismic design

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 American code (AASHTO, 2007) and Canadian code (CAN/CSA-S6-06) is compared for the stated performance objectives.

Economic impact on seismic performance requirement for bridges is incorporated in the life cycle cost (*LCC*) formulation taking into account the repair cost as well the user cost. Through the example of a two-span bridge, the estimation of optimal performance requirement is presented and the importance of user cost in such a calculation is highlighted.

2. SEISMIC PERFORMANCE OF BRIDGE PIERS

2.1. Performance Limit States

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).

Table 1 Performance Limit States

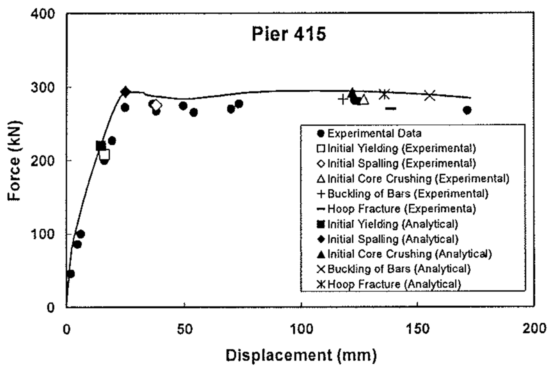
Limit states (LS)	Operational performance level	Post earthquake serviceability	Qualitative performance description	Quantitative performance description	Repair
1A	Fully Operational	Full service	Onset of hairline cracks	Cracks barely visible	No repair
1B			Yielding of longitudinal reinforcement	Crack width <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 ϵ_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 $\epsilon_c=\epsilon_{cc50}$ (initial core crushing) $\epsilon_c=\epsilon_{cu}$ (fracture of hoops) $\epsilon_s>0.06$ (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					

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.

2.2 Analytical Model for Seismic Performance Assessment

An analytical model for seismic performance assessment of bridge pier has been developed in a companion paper (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.

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, 815, and 430 have been reported herein (Figure 1). 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) (Figure 1).



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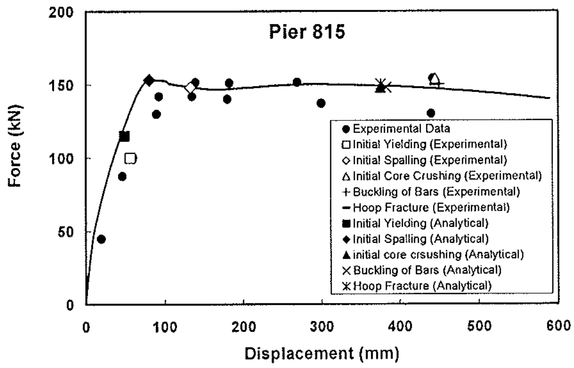

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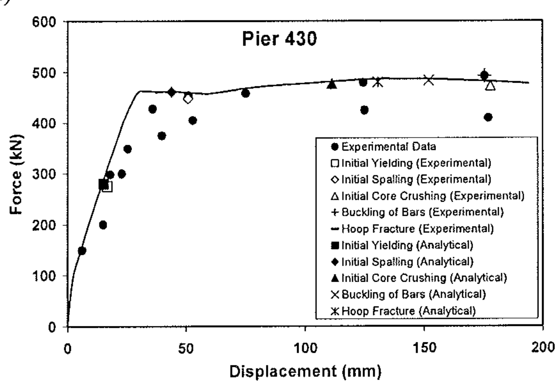

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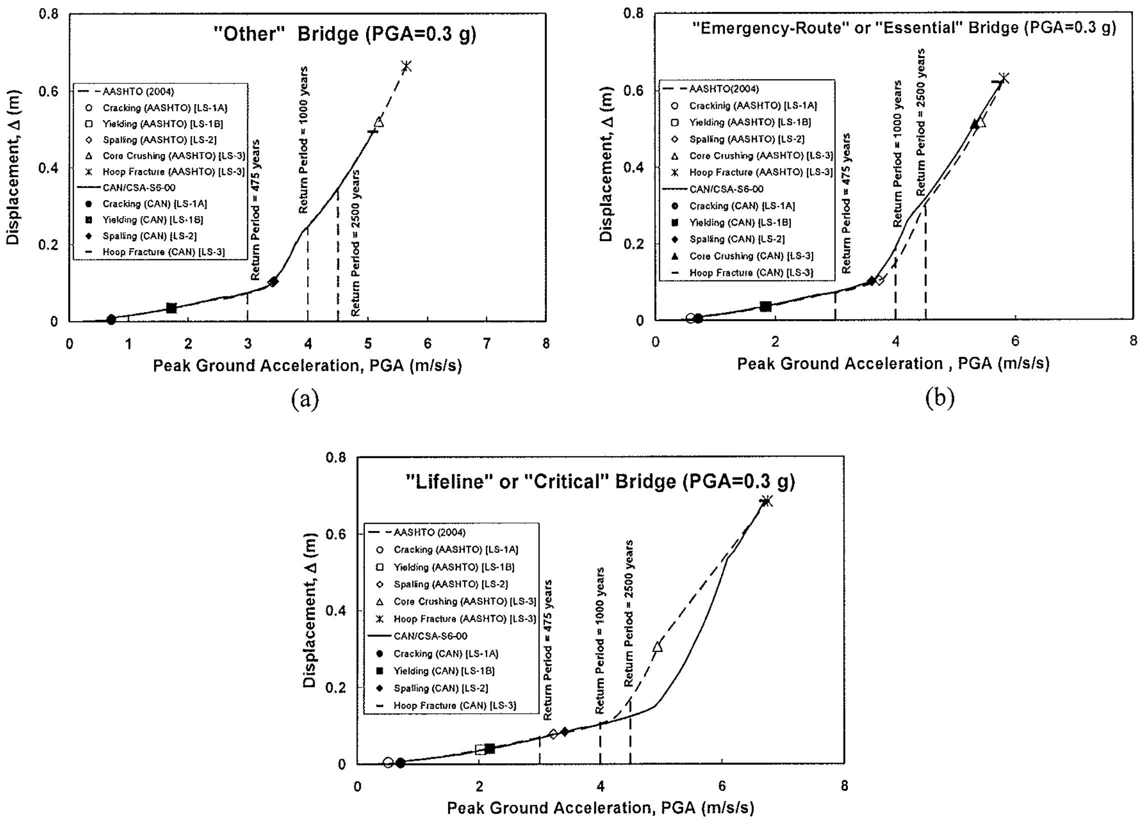
Figure 1 Experimental results compared with analytical predictions

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). As the model is able to adequately predict the performance of bridge piers, it is used for the performance estimation of the bridge piers designed according to current code provisions.

3. PERFORMANCE OF BRIDGE PIERS DESIGNED ACCORDING TO AMERICAN AND CANADIAN CODES

American code (AASHTO, 2007) and Canadian code (CAN/CSA-S6-06, 2006) have very similar seismic design philosophy. Both the codes classify bridges into three importance categories: lifeline or critical, emergency-route or essential, and other bridges. Lifeline or critical bridges are generally those which carry or cross over routes that must remain open to traffic after the design earthquake (475 year return period) and also be useable by emergency vehicles for security or defense purposes after a large earthquake (2,500 year return period in AASHTO and 1,000 year return period in Canadian code). Emergency-route or essential bridges are those which carry or cross over routes that should be open to emergency vehicles for security or defense purposes immediately after design earthquake. However, other bridges should not collapse in the event of a large earthquake and should be useable immediately after a small to moderate earthquake. The two codes propose two different set of design requirements taking into account the importance of the bridge. AASHTO (2007) provides a unique seismic level with a 475 year return period, but uses different response modification factor depending on the importance of the bridge, reflecting different performance levels expected. For example R=1.5 for a single column supporting a critical bridge and R=3 for a single column supporting a bridge neither essential nor critical (other bridge). Canadian code has constant load reduction factor and defines an importance factor (I) that is 3 for lifeline or critical bridges and 1 for bridge neither lifeline nor emergency-route bridges. The seismic level is determined based on the level corresponding to the 475-year return period earthquake multiplied by the importance factor, I. It is interesting to compare those two approaches and check if the performance is in agreement with the stated design principles. For this purpose many bridge configurations have been studied. However, due to space restriction, only a simple two span bridge supported by a single pier is presented herein. The bridge has a span length of 20 m; height of the single column piers is 9 m; superstructure unit weight is 150 kN/m. The design peak ground acceleration is 0.3g, corresponding to a return period of 475 years. The bridge is designed considering it as a lifeline bridge (or critical bridge) and as other bridge. 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*). On those curves, the performance limit states of the piers are also reported. It can be observed that code based design for "Other Bridge" is overly conservative (Figure 2a) since for 0.3g (the design *PGA*), the piers has a lot of excess capacity beyond the 475 year return period earthquake. No significant difference between Canadian code and American code is observed. This is expected as both codes have similar design spectra and force reduction factor R. On the contrary, performance of "Critical or Lifeline Bridge" falls short of meeting the required fully operational performance objectives (Figure 2c), since the bridge may not be fully operational for 475 year return period earthquake event. Moreover, considerable damages occur for large earthquake events (1,000 year or 2,500 year return period). However, "Emergency-Route" or "Essential" bridge meets the specified objectives in the design codes (Figure 2b). Canadian Code and American Code have significant differences in terms of safety for large earthquake events. Similar observations have also been found for bridges designed for different ground motion intensity levels and also for bridges supported by pier bent (Guiziou et al., 2006). Hence, proper evaluation of response modification factor and importance factor is essential and is a subject of further research.



(a) (b)

(c)

Figure 2 Seismic vulnerability of bridge piers

4. LIFE CYCLE COST ANALYSIS AND OPTIMAL PERFORMANCE

4.1. Analytical Model for Seismic Performance Assessment

It has been demonstrated in the previous section that code based design requirements cannot ensure adequate performance of bridges considering the design principles stated in the codes. The objective of this paragraph is to check, based on life cycle cost consideration, what the design principle should be.

Increase in seismic performance will increase initial construction cost, but in return will limit post-earthquake repair cost and economic impact. In order to design the bridge economically, it is important to design it with due consideration to total life cycle cost (*LCC*) for balancing initial construction cost and expected cost occurring within the design life of the bridge. *LCC* of a bridge consists of initial construction cost, maintenance cost, failure cost (repair cost, user cost, social and environmental cost, and so on), and cost of loss of lives and injuries. Maintenance cost has been omitted as it is not directly related to earthquake damage. Moreover, data on cost of loss of lives and injuries are scarce and are not considered in this study. Hence, the *LCC* considering seismic risk can be calculated as:

$$\mathrm{LCC}=\mathrm{C_i}+\sum_{n=1}^{n=N}\ \sum_{J=1}^{j=k}\ \mathrm{P_{nj}}\times\mathrm{C_j}\times e^{-\lambda t} \qquad (1)$$

where C_i is initial construction cost of new or retrofitted bridge; n is the severe loading occurrence number, N is the total number of sever loading occurrence; j is the number of limit state considered; k is the total number of limit states; P_{nj} is the probability of j^{th} limit state being exceeded given the i^{th} occurrence of earthquake; C_j is the cost of damage and user cost in present value due to j^{th} limit state; $e^{-\lambda t}$ is the factor accounting for discount over time period t; and λ is the constant discount rate usually ranging from 2 to 5%.

Considering only four limit states (Table 1), and assuming that the limit state probability Pnj does not change with time (i.e. ignoring the deterioration capacity of the structure with time), *LCC* can be calculated as (Wen and Kang, 2001):

$$LCC=C_i+ (C_{1A}P_{1A}+C_{1B}P_{1B}+C_2P_2+C_3P_3) \times (1-e^{-\lambda t}) \quad (2)$$

where C_j=limit state (1A, 1B, 2 and 3) failure cost and user cost; P_n is the annual probability of earthquake occurrence. Limit state failure cost includes repair cost of the damage and user cost. User cost can be defined as the sum of time cost and energy consumption cost due to the detour or closure of the road.

4.2. life Cycle Cost of an Example Bridge

It is difficult to calculate optimal design principle for a typical bridge that is applicable to all cases since it has to be adjusted on a case by case basis. For the sake of simplicity and to illustrate the *LCC* methodology, 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.

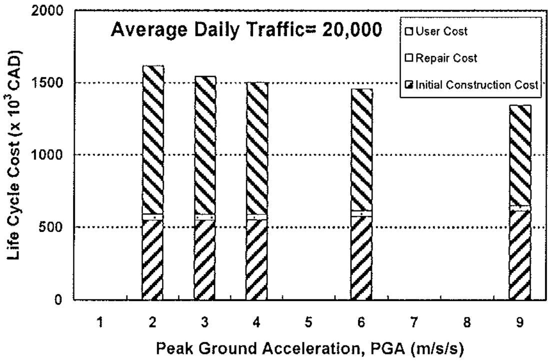
The bridge is designed for different peak ground acceleration levels. Construction cost of the pier and foundation are calculated based on material cost and labour cost. However, the construction cost of the superstructure is considered as a constant value of 400,000 CAD. The assumption of constant cost for superstructure is reasonable as it does not vary significantly with the level of design earthquake ground motion. In fact, bridge piers are the primary structural element that almost entirely carries the earthquake induced load.

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 very 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).

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.

Uniform hazard spectra have been developed by Geological Survey of Canada corresponding to four hazard levels: 40%, 10%, 5% and 2% probability of exceedance in 50 years. This represents annual exceedance frequency of 0.01, 0.0021, 0.001, and 0.0004, respectively. 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. 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 (consists of a large portion of total construction 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.



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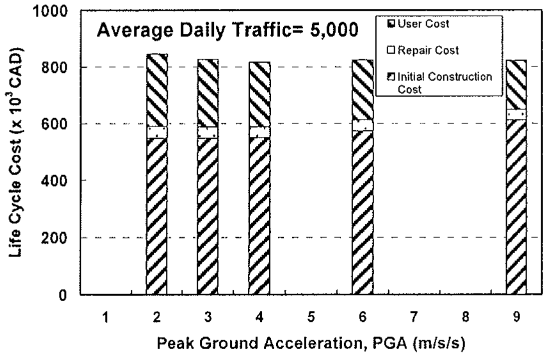

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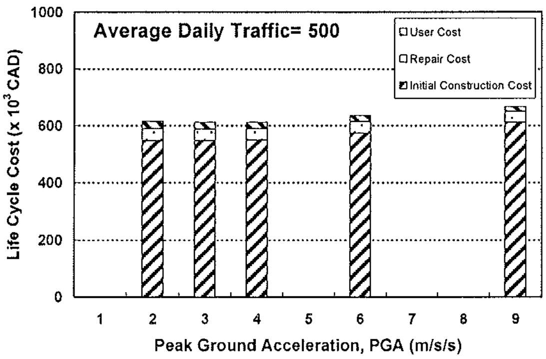

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Figure 3 Life-cycle cost analysis of a two-span bridge

It is evident that for a bridge located in a busy roadway, life cycle cost decreases when the bridge pier is designed for higher acceleration level (Figure 3a). 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 3b). Whereas in a remote place, life cycle cost is minimum at around 0.3g, which corresponds to 475-year return period earthquake event (Figure 3c). 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 (Figure 3). Hence it is prudent to design the bridge pier for higher earthquake acceleration level when the bridge is located in a busy roadway. In contrast, the bridge piers can be designed for design *PGA* level when it is located in places with limited traffic. 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. Estimation of expected life cycle cost of typical bridges is the subject of ongoing collaborative research between Wollongong University and Sherbrooke University.

5. CONCLUSIONS

A systematic 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 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.

The developed methodology for life cycle cost estimation has been applied to a simple two span bridge supported by a single pier. It has been observed that life cycle cost of a bridge depends largely on the user cost. If the bridge is located in a busy roadway, it is economical to design the bridge for a higher level of earthquake ground motion.

This study should be extended and results could help bridge owners to decide rationally the level of earthquake for which their structures should be designed.

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