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Seismic Performance of Bridges Designed According to AS 5100

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ABSTRACT

Bridges are the critical components of a nation's transportation system, as closure of an important bridge in the event of an earthquake can disrupt the total transportation network. In Australian standard for bridge design, ASBD (AS 5100-2004), consistent with other major bridge design codes (for example, AASHTO in the USA and CAN/CSA-S6 in Canada), bridges are classified according to their importance levels. The anticipated performances (performance objectives) of the bridges in small to moderate (Return Period, RP= 100 years), design level (RP= 500 years) and large (RP= 2500 years) earthquake events have been specified in major bridge design codes, although not explicitly stated in ASBD for bridge design. It is believed that similar performance objectives should also be anticipated for the bridges designed for different importance levels according to ASBD. However, there appears to be no requirement in the code to check whether such multiple performance objectives have been achieved for the designed bridges. Also, no engineering parameters have been assigned to the anticipated performance objectives. This paper correlates seismic performance objectives (both qualitative and quantitative) with engineering parameters which are based on the data collected from available experimental investigations and field investigations from recent earthquakes. A simple methodology has been developed and validated with experimental results for assessing the performance of bridges designed according ASBD. It has been found that the design rules prescribed in ASBD do not guarantee that intended multiple seismic performance objectives can be obtained. Implicit seismic design rule in the form of Performance Response Modification Factor (PRMF) has been outlined for performance based seismic design of bridges. The implicit design rule has the potential for further development in order to be incorporated in the next generation ASBD.

Keywords: Bridges, design codes, seismic design, seismic evaluation, performance objectives

INTRODUCTION

Bridges are essential part of the transportation system worldwide, as the closure of important bridges due to damage or collapse in the event of an earthquake can disrupt the total transportation network. Even so, engineering community paid inadequate attention on the seismic design and performance of bridges until the collapse of several highway bridges in the 1971 San Fernando, California, USA, earthquake causing significant economic losses.

Extensive research investigations have been conducted on the seismic behaviour of reinforced concrete bridges afterwards. Also, past earthquakes revealed several deficiencies in the design and detailing of bridges. Significant improvements in both design practice and analytical methods have been achieved (Saiidi, 2011). Some of these new developments have already been incorporated in the design codes. Nonetheless, future performance of engineered bridges when designed according to the current code provisions is not known with sufficient confidence (Sheikh and Legeron, 2012).

The seismic design rules in Australian Standard for Bridge Design (ASBD) were developed based largely on force-based design approaches. The seismic force level corresponding to elastic response to a design acceleration response spectrum for a soil site class is calculated based on an estimate of elastic stiffness of the structure. This elastic force is then modified by a Structural Response Factor, R_f, for an assumed ductility capacity of the bridge pier and an importance factor, I, for the expected performance in an earthquake. Current ASBD classifies bridges into three different types (Type I, II and III), which is similar to other international bridge design codes. Type III bridge is comparable with life-line / critical bridge in AASHTO (2007), CAN/CSA-S6 (2006) and EC8 (2004). Similarly, Type I and Type II bridges are comparable with emergency-route / essential bridge and other bridges, respectively. Importance factor (I) for Type I and Type II bridges is 1.0 and for type III bridges is 1.25. It is noted that I-factors suggested in ASBD are significantly lower than the recommendations in major bridge design codes (AASHTO, 2007; CAN/CSA-S6, 2006; EC8, 1994). Although in major seismic design codes expected performances of bridges in future earthquake events have been specified (Table 1), no such specification has been provided in ASBD. It is believed that similar multi-level performance objectives should also be anticipated for the bridges designed for different importance levels according to ASBD.

Table 1. Performance requirements in major seismic bridge design codes

Return period	Bridge		
	Lifeline (Type III)	Emergency-route (Type II)	Other (Type I)
Small to moderate Earthquake (100-year return period	All traffic Immediate use	All traffic Immediate use	All traffic Immediate use
Design earthquake (500-year return period)	All traffic Immediate use	Emergency vehicles Immediate use	Repairable damage
Large earthquake (2500-year return period)	Emergency vehicles Immediate use	Repairable damage	No collapse

The current design rules in ASBD do not ensure future performances of bridges. A single R_f-factor recommended in the code may generally be suitable for a single level of design (a particular performance for the chosen level of earthquake event); multi-level design requires a set of R_f-factors. Moreover, the arbitrary chosen I-factor should be dependent on the seismicity and properties of the bridge. Like other major bridge design codes, ASBD does not require design engineers to explicitly check the seismic performance of the designed bridges. Although damages are anticipated in the future earthquake, as indicated in Table 1, there is no consideration of the extent of the damages in the design procedure. Moreover, the arbitrary chosen I-factor should be dependent on the seismicity and the properties of the bridge. The weakness of the design rules in ASBD has been highlighted in this paper.

This paper first correlates performance objectives with engineering parameters based on published experimental results. A simplified methodology for seismic assessment of bridges has been outlined and verified with the results from experimental investigations. The seismic performance of bridges designed according to current ASBD has been discussed based on the design of a typical three-span bridge. It has been revealed in this paper that design rules adopted in ASBD do not satisfy the anticipated performance objectives as in Table 1. An implicit seismic design rule has been outlined for performance based seismic design of bridges in Australia.

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 earthquakes (Table 1). The performance requirements stated in the design codes are just descriptive. Table 2 provides actual performance levels 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 2. Qualitative and quantitative performance levels correlated with engineering parameters and repair techniques

Limit states (LS)	Operational performance level	Post earthquake serviceability	Qualitative performance description	Quantitative performance description	Repair
1A			No cracks	$\sigma_c = f_{cr} = 0.4 \sqrt{f'_c}$	No repair
1B	Fully Operational	Full service	Few cracks that can be easily repaired and with no consequence on serviceability	$\sigma_s = f_{sy}$	Limited epoxy injection
2	Delayed Operational	Limited service (emergency vehicle only)	Initiation of inelastic deformation; onset of concrete spalling; development of longitudinal cracks	$\epsilon_c = -0.004$ $\epsilon_s = 0.007$ crack width= 2 mm	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	$\epsilon_c = \epsilon_{cc50}$ (initial core crushing) $\epsilon_c = \epsilon_{cu}$ (fracture of hoops) $\epsilon_s = \epsilon_{su} = 0.07$ (longitudinal reinforcement fracture) $\epsilon_s = \epsilon_{scr}$ (onset of buckling)	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=average tensile strain in longitudinal reinforcement, ε_{su}=tensile strain at fracture; ϵ_{scr}= steel strain at onset of buckling of longitudinal bars

Both qualitative and quantitative performance levels are described in Table 2 and are associated with engineering parameters. Up to the limit state 1A, the response is elastic with small displacement amplitude. At this limit state no cracking of concrete is expected to occur and no post-earthquake repair is necessary. Beyond limit state 1A and up to limit state 1B, the concrete may crack, but the damage should be minor and easily repairable. There might be few crack openings, but the capacity of the pier shall not be affected noticeably and the bridge shall remain fully operational. Moderate structural damage may occur up to limit state 2. However, the bridge may remain functional for emergency and defence/security vehicles only. Up to performance level 3, significant structural damage is expected to occur but the bridge should not collapse. The bridge will not be useable after the earthquake and extensive repairs may be required. Such repair may not be always economically feasible and reconstruction might sometimes be necessary.

The above descriptions of performance of a bridge have been summarized in Figure 1. It can be observed that the region between limit state 2 and limit state 3 has been divided into two regions: Repairable damage and Reconstruction. Experimental results for engineering parameters to clearly distinguish between repairable damage and reconstruction are not currently available. Therefore, a displacement capacity midway between limit state 2 and limit state 3 has been chosen as the limiting deflection between repairable damage and re-construction (Figure 1).

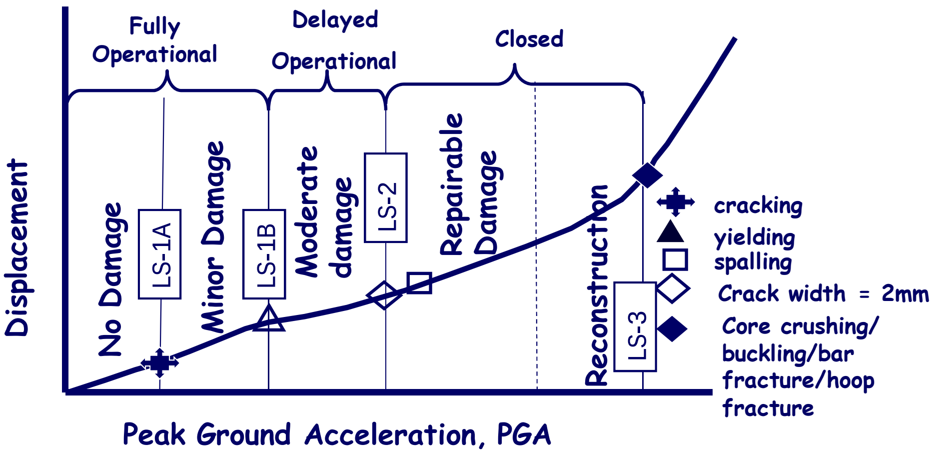

Figure 1. Post-earthquake serviceability and performance limit states

SIMPLIFIED METHOD FOR SEISMIC PERFORMANCE ASSESSMENT OF BRIDGES

A simplified seismic assessment methodology, which has incorporated non-linear monotonic static analysis approach, has been developed in Sheikh and Legeron (2012) for seismic performance assessment of bridge piers based fully on displacement principle. The method adopts local approach for direct comparison of the performance of bridge piers with performance limit states described in the earlier section. The method takes into account the non-linear behaviour of materials including the confinement effects from lateral reinforcement. The displacement components that contribute to the total tip displacement include bending and shear deformation along the pier length together with deformation due to slip of the longitudinal reinforcement at the joint. A brief description of the developed model has been presented blow.

(a) Non-linear material model for reinforced concrete

Realistic constitutive model for highly non-linear reinforced concrete model is complex, as the nonlinearity arises from the constituent concrete and reinforcement should be appropriately combined. The non-linear model of reinforced concrete consists of constitutive model of concrete and reinforcement.

The realistic constitutive law of concrete needs to take into account the effect of confinement on the overall stress-strain behaviour of concrete. In the developed model, the uniaxial confined concrete model proposed in Légeron and Paultre (2003) has been adopted as the constitutive law of concrete.

The non-linear reinforcing bar model proposed by Gomes and Appleton (1997) has been chosen due to its simplicity and accuracy in predicting the section behaviour of piers. Gomes and Appleton (1997) model takes into account the effect of inelastic buckling of longitudinal reinforcing bars in a simplified way based on the plastic mechanism of buckled bar. It is noted that when a bar is subjected to cyclic load, its maximum strength is less than the maximum strength observed in monotonic tensile tests. Ultimate limit strain of the bar has been considered as 0.07, as described in the previous section.

(b) Modelling for sectional behaviour

The sectional behaviour (moment-curvature) of the columns is modelled in the computer program MNPhi (Paultre, 2001) adopting the constitutive laws of concrete and reinforcing bars as discussed above. By assuming the strain profile, which complies with the assumption that plane sections remain plane, the stress in each layer and in the reinforcement is calculated. Based on the calculated axial force, the strain profile is then updated which converges with the applied axial force.

c) Modelling for force-displacement behaviour

The displacement at the top of the pier which is fixed at the base is considered to be consisted of displacement due to bending (elastic and inelastic) along the length of the column, shear displacement along the length of the column and displacement due to fix end rotation for slip of the longitudinal reinforcement. Using the flexibility approach, the tip displacement of the pier is the sum of these three components:

$$\Delta_{top}=\Delta_b+\Delta_s+\Delta_{slip} \quad =(\Delta_e+\Delta_p)+\Delta_s+\Delta_{slip} \qquad \text{Eq. (1)}$$

Where Δ_e is the elastic displacement, Δ_p is the plastic displacement, Δ_s is the shear displacement and Δ_{slip} is the slip displacement. Detailed description of the calculation procedure for Δ_p, Δ_s, and Δ_{slip} can be found in Sheikh and Legeron (2012).

(d) Comparison with experimental investigations

The above modelling approach for force-displacement response of bridge piers has been validated with the experimental investigations of all the bridge piers in Lehman et al. (2004). The tested columns had circular cross sections (610 mm diameter) and were reinforced with well-distributed longitudinal reinforcement and closely spaced spiral reinforcement. The test variables included aspect ratio (L/D=3 - 8), longitudinal reinforcement ratio (ρ_l=0.75 – 2.8%), spiral reinforcement ratio (ρ_s=0.35 – 0.87%), axial load ratio (n=0.1 - 0.2), and the length of the well-confined region. The comparison between experimental results and analytical investigation for two columns has been shown in Figure 2.

It can be observed that the analytical model predicts the experimental result with very good accuracy. Also, performance points, as needed for the performance evaluation, have been well predicted. All the limit states have been predicted with an average difference of only 10%. As the analytical model not only predicts the force displacement response but also predicts quite well the performance points, the modelling technique has been applied for the performance evaluation of bridge piers design according to ASBD in the following sections.

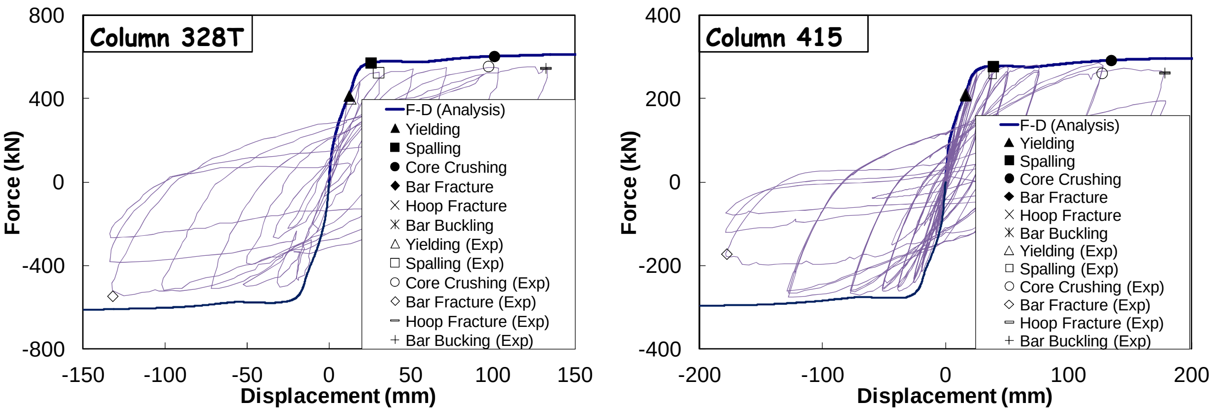

Figure 2. Comparison between experimental results and analytical investigation

(e) Seismic performance of bridge piers

Seismic performance of bridge piers designed according to ASBD has been evaluated in terms of the peak ground acceleration (PGA)-displacement response, based on substitute structure approach (Shibata and Sozen, 1976), using the computed force-displacement response. The secant stiffness and the effective period of the bridge pier at every point on the force-displacement response have been calculated. The force level and the peak ground acceleration corresponding to the spectral ordinate (damped response spectrum after application of DMF) and period have been calculated and represented as PGA-displacement response of the bridge pier. Performance points in terms of engineering parameters (Table 2) have been superimposed in the PGA-displacement response of the bridge pier (Figure 1). Such diagram clearly identified the damages and the corresponding earthquake ground motion in terms of the PGA of the ground motion.

It is noted that the methodology described above is to identify the local damage parameter (for bridge piers). However, such damage parameters may reasonably indicate the seismic damage states of bridges supported only by piers.

SEISMIC PERFORMANCE OF A TYPICAL BRIDGE DESIGNED ACCORDING TO AUSTRALIAN STANDARD FOR BRIDGE DESIGN

The application of the developed seismic performance evaluation methodology has been demonstrated for a typical 3-span highway bridge (S10-LD5-T10-N10) (Table 3). The lengths of end spans and mid-span are 28 m and 35 m, respectively. The bridge deck is 200 mm thick which is covered by a 65 mm overlay with two concrete barriers. The road supported by the bridge has two 3.6 m lanes and 2.4 m shoulders. The bridge is supported by two single piers with pier cap at the top. The bridge pier has been designed according to the design guidelines in ASBD. The design strength of concrete is taken as 40 MPa with longitudinal reinforcement of 1.0%. The transverse reinforcement ratios at the plastic hinge region and other regions have been calculated as 0.96% and 0.35%, respectively. Based on an iterative procedure such design corresponds to an AI value (PGA x Importance Factor) of 0.16g. It is noted that the bridge is supported by piers only and hence the performance of bridge pier reflects the performance of the bridge.

Table 3. Properties of the three span bridges

Case Number	Type of pier	L/D	T (s)	D (m)	$n=N/A_g f'_c$	W_s (kN)
S10-LD5-T10-N05	Single	5	1.0	1.0	0.05	4800
S10-LD5-T10-N10	**Single**	**5**	**1.0**	**1.0**	**0.1**	**4800**
S10-LD5-T10-N20	Single	5	1.0	1.0	0.20	4800

L/D is the aspect ratio (length/Diameter) of the bridge pier, T is the vibration period of the bridge/bridge pier, n is the axial load level (N= axial load, A_g= gross area of the pier, f'_c is the strength of concrete) and W_s is the seismic weight of the pier

Force-displacement and PGA-displacement response of the typical bridge have been shown in Figures 3 (a-b). The actual yield strength/design yield strength has been calculated as 1.6, which is considered relatively very high. However, the critical evaluation of the overstrength factor for the design of bridge in ASBD is considered beyond the scope of the paper. The performance of the bridge in different earthquake events can be evaluated once the PGA value of such earthquake event is known. The PGA values other than design earthquake events are usually expressed as ratios of the PGA for design earthquake events termed herein as normalised PGA (or probability factor, k_p, as in AS 1170.4, 2007). The k_p values are 0.5, 1.0, and 1.8 for earthquakes of return periods 100, 500, and 2500 years, respectively (AS 1170.4-2007). In order to better represent the performance of the bridge in different return period of earthquake events, the response of the bridge in terms of normalised PGA (or probability factor) has been shown in Figure 8 (c-d). It can be observed from the figure that bridge failed to achieve the intended multi-level performance as described in Table 1. If the bridge is designed as Type I bridge, it fails the requirement of no collapse performance objective for large earthquake events (normalised PGA=1.8). If the bridge is designed as Type II bridge, it fails to meet the requirements of repairable damage for large earthquake events and emergency vehicle immediate use for design level earthquake events (normalised PGA=1.0). If the bridge is designed as Type III bridge, it fails to meet the requirements of emergency vehicle immediate use for large earthquake events and all traffic immediate use for design level earthquake events (normalised PGA=1.0). It is important to note that this performances been achieved taking into account the conservatism due to overstrengh factor in the design procedure.

The above mentioned performances can be expressed in terms of Damage Response Factor (DRF) which is the ratio of PGA for the damage state under consideration and the PGA at yield (Table 4). DRFs are calculated based on the highest displacement of the limit state being considered. DRF provide a means for direct comparison of actual performance of the bridge and the anticipated performance (Table 5). The value of $(R_f/I)_{design}$ x (normalised PGA/DRF) greater than 1 represents the noncompliance between design rules and seismic performance objectives listed in Table 1. It can be observed that none of the performance objectives listed in Table 1 has been achieved by the design procedure in ASBD (Table 5). It is important to note that the performance comparison presented in Table 5 does not consider the overstrength factor achieved in the design procedure. The results presented above indicate that the design procedure adopted in ASBD is highly unconservative and may yield highly variable levels of damage under design earthquake scenarios.

Table 4. Seismic performance of typical bridge

Limit States	PGA at limit state	Normalised PGA[1]	DRF
Minor damage (LS-1B)	0.077	0.473	1.0
Moderate Damage (LS-2)	0.135	0.835	1.8
Repairable Damage	0.216	1.33	2.8
Reconstruction (LS-3)	0.243	1.50	3.2

[1]Normalised with respect to design PGA= 0.16g

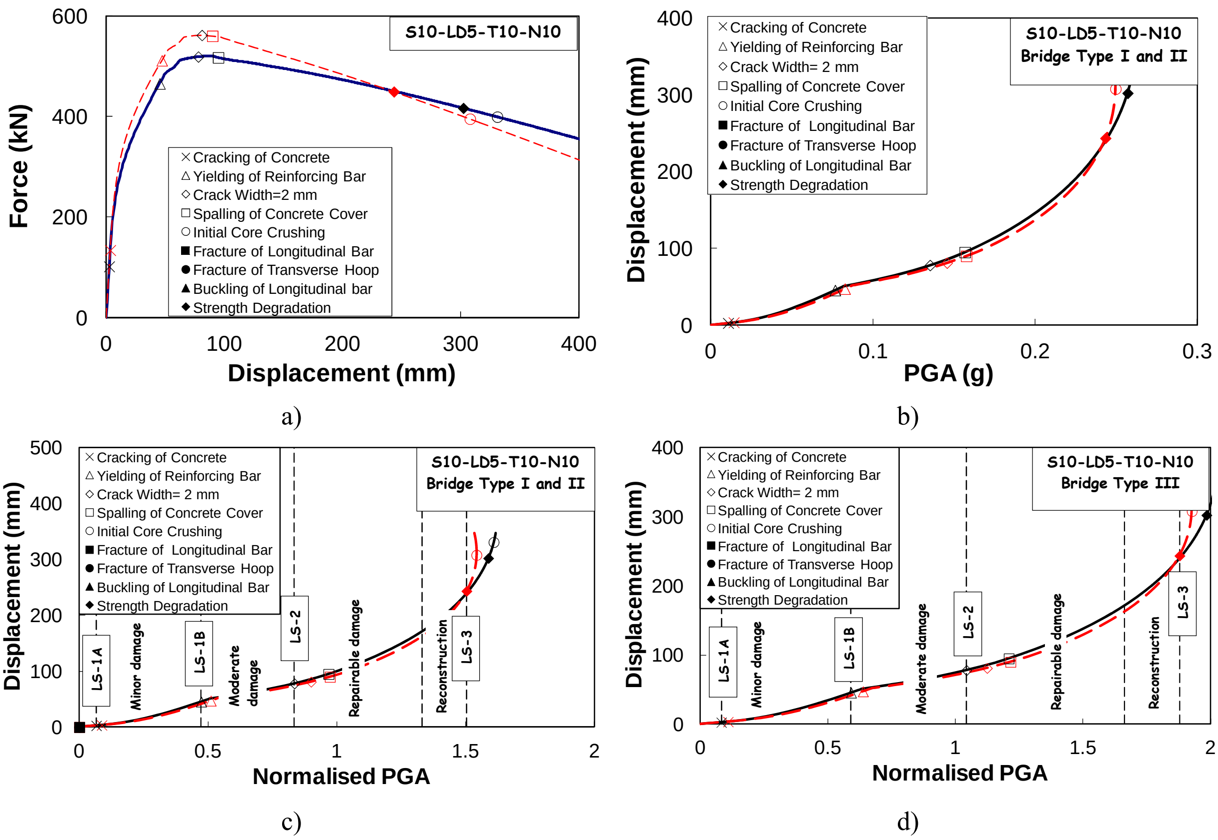

Figure 3. Seismic performance of bridges designed according to ASBD

The typical bridge described above has been modified to investigate the effect of axial load ratio (n) on the seismic performance of the bridge (S10-LD5-T10-N05 for n=0.05 and S10-LD5-T10-N20 for n=0.2) (Table 6). It can be observed that axial load level has significant influence on the seismic capacity of the bridge. The performance of the bridge is better under lower axial load level. The DRFs for bridge with n=0.05 for repairable damage and reconstruction are about 1.7 times of the DRFs for bridges with n=0.2. Hence, the level of axial load should be considered in the design procedure of the bridge.

Table 5. Actual seismic performance versus stipulated seismic performance for the typical bridge designed according to ASBD

Return Period	Normalised PGA	Damage response factor (DRF)			(R_f/I) design x (normalised PGA/DRF)		
		Line-line	Emergency-route	Other	Life Line	Emergency-route	Other
					R_f/I=2.8	R_f/I=3.5	R_f/I=3.5
Small to moderate earthquake(RP= 100 years)	0.5	1	1	1	**1.4**	**1.75**	**1.75**
Design earthquake (RP=475 years)	1	1	1.8	2.8	**2.8**	**1.94**	**1.25**
Large earthquake (RP= 2500 years)	1.8	1.8	2.8	3.2	**2.8**	**2.25**	**1.97**

Note: Bold fonts represent noncompliance (values >1) between design rules and seismic performance objectives in Table 1

Table 6. Damage Response Factor (DRF) for the case study bridges

Case Number	Type of pier	Moderate Damage	Repairable Damage	Reconstruction
S10-LD5-T10-N05	Single	1.8	3.4	4.0
S10-LD5-T10-N10	**Single**	**1.8**	**2.8**	**3.2**
S10-LD5-T10-N20	Single	1.7	2.1	2.5

Note: Bold fonts represent DRFs for typical bridge considered in this study

IMPLICIT DESIGN RULES FOR PERFORMANCE BASED SEISMIC DESIGN OF BRIDGES

Recent research efforts on the seismic design of bridges are towards the development of performance based seismic design of bridges, where designed bridges should meet multiple performance objectives under different earthquake scenarios. Multiple performance objectives, each pairing with seismic hazard level, require more complex design framework. A large number of analyses are required to develop such performance based seismic design framework. However, it should always be preferable for seismic design code to adopt simplified design guideline, as complex analysis techniques, training and resources may not be available in the design firms. Such a simplified design guideline has been outlined herein.

In Table 7 implicit PRMF factors have been presented for the typical bridge for different damage states or performance levels together with earthquake ground motion (PGA) for different return period events. To satisfy all the performance levels for different design earthquake ground motions, the simplest way is to adopt the minimum PRMF for the category of bridge under consideration. In the proposed implicit design rules, the importance of the bridge has already been considered in the PRMF. There is no requirement for any arbitrary importance factors. From Table 7, it can be observed that the design is dictated by large earthquake events with return period of 2500 years.

The implicit design rule presented herein clearly indicates the inclusion of seismicity of the area in the design and is supported by recent research investigations on performance-based seismic design and assessment of bridges. Such implicit design rules can also be obtained for other bridges once the DRFs of the bridges are known (Table 6). However, for the adoption of the simplified guidelines developed in this paper in the next generation ASBD, a large number of analyses are required with due consideration to different class of bridges with all possible variations in the material and geometric properties of the bridges. This forms the part of on-going research of the authors and collaborators.

Table 7. Implicit seismic design rules for the typical bridge

Earthquake Event	Normalised PGA	Damage response factor (DRF)			Performance Response Modification Factor (PRMF)		
		Life Line	Emergency-route	Other	Life Line	Emergency-route	Other
Small to moderate earthquake(RP= 100 years)	0.5	1	1	1	2.0	2.0	2.0
Design earthquake (475-year return period)	1.0	1	1.8	2.8	1.0	1.8	2.8
Large earthquake (RP= 2500 years)	1.8	1.8	2.8	3.2	**1.0**	**1.5**	**1.8**

Note: Bold fonts represent the design PRMFs

CONCLUSIONS

This paper correlates seismic performance objectives (qualitative and quantitative) with engineering parameters that were developed based on the data from experimental investigations and field investigations from recent earthquakes. A simplified assessment methodology has been outlined which is developed based on equivalent static (push-over) analyses procedures incorporating the substitute structure approach. The method has been fully validated with available experimental data. The developed method not only replicates force-displacement behaviour of the bridge piers but also replicates the performance points (engineering parameters) with reasonable accuracy. It has been found that design rules in ASBD do not satisfy the intended multiple seismic performance objectives. It has also been found that the design rules adopted in Australian Standard for Bridge Design (ASBD may yield highly variable levels of damage in an earthquake event. This paper presents the seismic performances of bridges in terms of damage response factor (DRF) for the three bridges designed according to ASBD. Implicit seismic design rules for a typical bridge have been outlined which are based on damage state (or DRF) and seismicity of the area. Such implicit design rule has the potential for further development in order to be incorporated in the future seismic design codes.

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