SEISMIC PERFORMANCE COMPARISON BETWEEN FORCE-BASED AND PERFORMANCE-BASED DESIGN OF A HIGHWAY BRIDGE

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Abstract: Canadian Highway Bridge Design Code (CHBDC) 2014 initiated Performance-Based Design (PBD) in Canada. For Lifeline bridges and irregular Major Route bridges, PBD has to be used to explicitly demonstrate structural performance. Regular Major Route bridges can be designed by using FBD or PBD method. In this study, a concrete bent highway bridge is designed by using both FBD and PBD based on CHBDC 2014, and FBD based on CHBDC 2006. Soil-structure interaction is incorporated by using p-y curves in the design and analysis. Dynamic time-history analyses are performed to assess the seismic performance. The assessment is based on the maximum strain limits from CHBDC 2014.

1. Introduction

Transportation systems are essential prerequisite for economic development, and bridges are the critical elements in a transportation system. Seismic design is one of the most challenging parts in the bridge design process. Traditionally, bridges are only designed by using Force-Based Design (FBD) method. Either single-mode or multi-mode spectral method can be used in this procedure. For simple bridges, a simplified single-mode method called uniform-load method is often used. However, it was demonstrated that the current FBD has many shortcomings (Priestley et al., 2007). The major limitation in the FBD method is that it cannot explicitly relate to the performance of the bridges as there are many uncertainties in achieving the expected level of performance. The second limitation of FBD is force-reduction factor R, which is utilized to scale down the seismic force. The R factor can vary significantly for similar type of structures with different geometry. It is based on ductility capacity and over-strength for a given structure type. Utilizing one R factor for different elements may not be appropriate. The third limitation is that design seismic force is applied to the structures with unchanged stiffness, which indicates that the elements of the structure are subjected to yield at the same time. In reality, seismic force distribution is also affected by the deformed shape of the structure. The stiffness of a structure is not constant as what is assumed in FBD, as it changes with its deformation. To overcome the shortcomings of FBD and explicitly demonstrate structural performance, researchers have developed Performance-Based Design (PBD).

PBD has been proposed by many researchers in the past several decades (FEMA, 1997; Poland et al., 1995; Priestley, 2000). PBD has been adopted by some of the design codes and guidelines such as CHBDC 2014 (CSA, 2014). By using PBD, owners are allowed to select target performance levels and designers are able to control the performance explicitly. The performance criteria are usually based on maximum drifts, residual drifts, displacement and material strains. Reza et al. (2014) have proposed equations to estimate the cracking, crushing and yielding displacement of bridge piers. This can be of great use for the preliminary PBD. Rather than using forces like FBD, PBD uses criteria that can be directly measured and observed.

There are many applications of PBD in large projects aiming to reduce the uncertainty of the designs (Marsh and Stringer, 2013). PBD relates scientific designs with performance explicitly so that decision makers can better distribute investments. In terms of both life safety protection and limiting damages, PBD may have more advantages over FBD. FBD was frequently failed to provide satisfactory performance in major earthquakes. Even if the FBD achieved the goal of protecting life safety, the costs of repair were unexpectedly high after earthquakes like 1994 Northridge earthquakes (Ghobarah, 2001). In PBD, it is believed that displacement and deformation determine seismic damages, rather than strength and capacity (Marsh & Stringer, 2013). Therefore, PBD can be a prominent method for future design of bridges. The research of PBD is still in a development stage, with prominent results coming constantly.

To compare the differences between FBD and PBD approaches from CHBDC 2014, a bridge was designed using both methods. A comparison between CHBDC 2014 and CHBDC 2006 was also performed. The key comparison between these two methods helps establish the expected performance of bridges at various levels of design earthquakes. This research determines whether FBD is more conservative or un-conservative in comparison with PBD. In CHBDC 2014, there are many descriptive performance criteria. The quantitative criteria include material strains and residual seismic capacities. However, the code has no exact definition of residual seismic capacities. Although there are many performance criteria in CHBDC 2014, this study is only based on strain criterion. The damage states and strains from CHBDC 2014 are briefly described in Table 1.

Table 1 – Performance Criteria (CHBDC, 2014)

Level	Service	Damage	Criteria
1	Immediate	Minimal Damage	Concrete compressive strains (ε_c) $\leq$ 0.004 and steel strains (ε_{st}) $\leq$ yield strain (ε_y).
2	Limited	Repairable Damage	Steel strains (ε_{st}) $\leq$ 0.015.
3	Service Disruption	Extensive Damage	Confined core concrete strain (ε_{cc}) $\leq$ concrete crushing strain (ε_{cu}). Steel strains $\leq$ 0.05.
4	Life Safety	Probable Replacement	Bridge spans shall remain in place but the bridge may be unusable and may have to be extensively repaired or replaced.

2. Case Study Description

The case study presents a multi-span concrete bent bridge, which is categorized as a regular Major Route bridge. The total span length is 100 meter and the width is 40 meter. There are three bents as piers and two bents as abutments. Each bent are similar in geometry and has eight columns. The columns are supported by single piles since the shallow soil is weak. The soil-structure interaction is a critical part of the design. In practice, the interaction between soil and structure is usually simulated by using p-y curves due to its simplicity. (Dash et al., 2008). In p-y curves, p stands for lateral resistance force per unit pile length from soil, and y stands for lateral displacement of piles. A typical p-y curve is shown in Figure 3, where the soil loses both strength and stiffness with the increase of displacement. In this study, soil-structure interaction is considered in the bridge design and performance assessment. In the design phase, the bridge model was built in SAP2000 (CSI, 2010) and the soil-structure interaction was simulated by using a series of linear p-y springs. The finite element model of the bridge is shown in Figure 1. Site-specific response spectra were used for the design, and the spectra accelerations are shown in Figure 2.

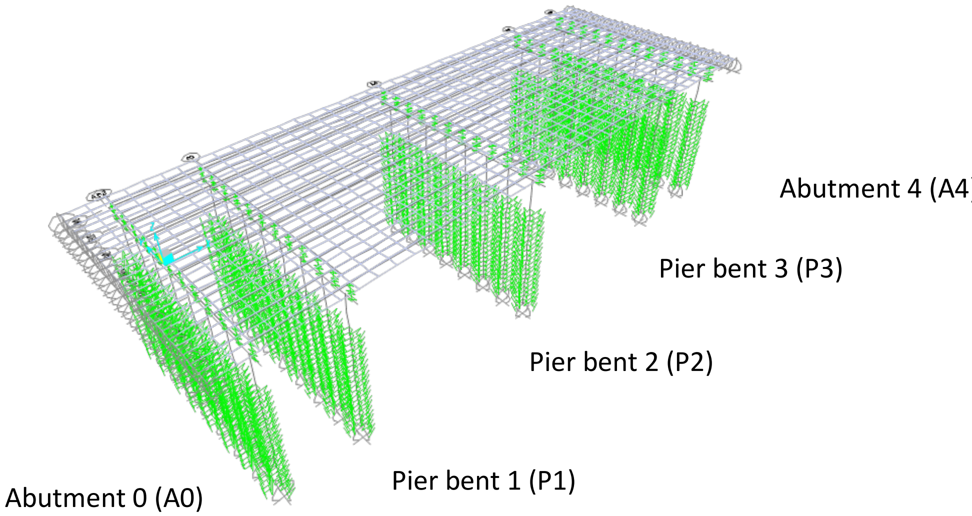

Fig. 1 – Finite element model in SAP2000

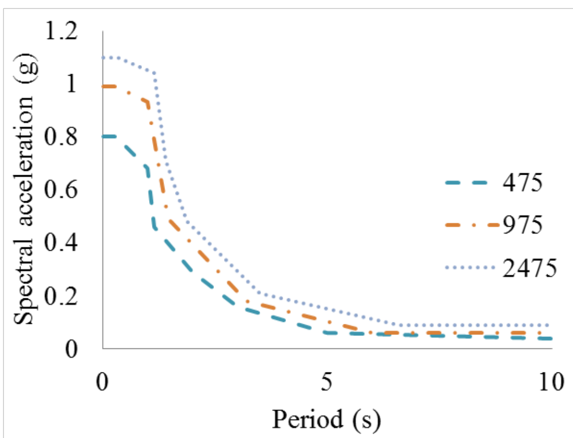

Fig. 2 – Response spectra

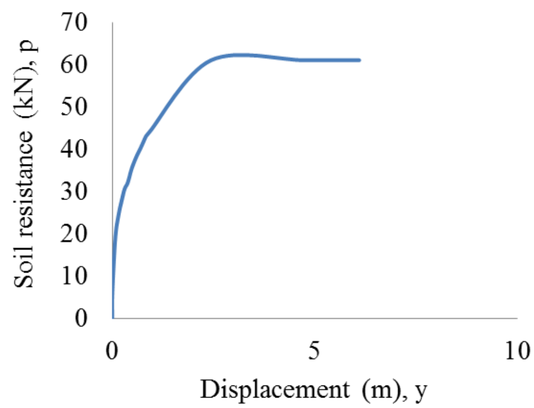

Fig. 3 – Typical p-y spring from field test

In this study, a total of 3 designs are carried out and assessed. FBDs were carried out as per CHBDC 2006 (CSA, 2006) and the CHBDC 2014 (CSA, 2014) respectively, which are denoted as D1 and D2. One PBD was conducted as per the CHBDC 2014 (CSA, 2014), which is denoted as D3. In the FBD, an importance factor of 1.5 shall be used as per CHBDC 2014. The three design results are shown in Table 2 and Fig 4.

Table 2 – Design cases

Case No.	Design method	Design Code CHBDC	Column diameter (m)	Column Longitudinal reinforcement ratio	Return period (years)	Longitudinal period (s)	Transvers period (s)
D1	FBD	2006	0.914	1.9%	475	1.984	1.787
D2	FBD	2014	0.914	2.7%	2475	2.244	2.068
D3	PBD	2014	1.2	5.3%	475	1.598	1.362
					975	1.621	1.422
					2475	1.700	1.474

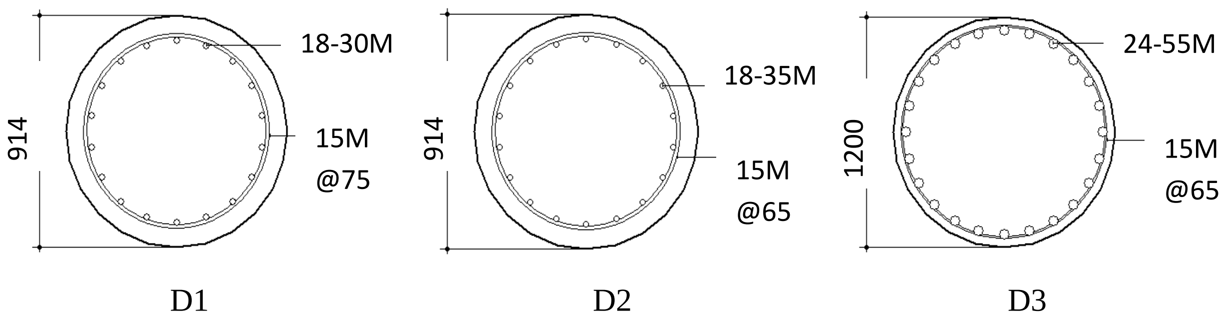

Fig. 4 – Column section

Comparing the two FBDs, D2 is more conservative since it was designed to 1/2475 year event with an importance factor of 1.5. This is straightforward because CHBDC 2014 aims to improve the seismic performance of structures. This finding also applies to other similar design cases. On the other hand, the longitudinal reinforcement of D3 is extremely high, although the diameter of the column was increased to 1.2m to reduce displacement demands. This is due to the requirement from the CHBDC 2014 that steel strains shall not exceed yield at 1/474-year event.

3. Performance Assessment Based On Time-History Analysis

After designing the bridge based on three different approaches, time-history analyses were carried out to evaluate their seismic performance. The comparing criteria were material strains based on CHBDC 2014. It should be noted that although a conservative structure is more likely to protect life safety, a good design does not mean to be extremely conservative. An extremely conservative design may not only result in high cost and being impractical to construct, it may also have more negative environmental impact. Based on this philosophy, D1, D2 and D3 are compared based on time-history analyses.

7 earthquake records were selected from The Canadian Association for Earthquake Engineering (Naumoski et al., 1988) for time-history analyses. The earthquake records were scaled based on site-specific response spectra. Acceleration loads were applied in both horizontal directions at the same time. Tables 3 to 5 present maximum strains from time-history analyses. In the tables only the results from the first 3 earthquake records are shown. Table 6 shows the damage states of the three designs determined from average strains of time-history analysis.

It was concluded that D1 and D2 failed to meet the criteria at 1/475-years event. D3 met the criteria at all earthquake events and only reached repairable damage states at 1/2475-year event. However, D3 has an extremely high rebar ratio, which is 5.3%. Although the upper limit of rebar ratio from CHBDC 2014 is 6%, 5.3% rebar ratio may have some challenges in construction. This results in a waste of resources and an increase in the cost.

Table 3 – Maximum strains of D1 from time-history analysis

Return period (years)	Material Damage	Earthquake record number		
		1	2	3
475	Concrete	0.003	0.003	0.003
	Steel	0.006	0.006	0.006
	Damage	Repairable	Repairable	Repairable
975	Concrete	0.004	0.005	0.006
	Steel	0.01	0.009	0.01
	Damage	Repairable	Repairable	Repairable
2475	Concrete	0.015	0.006	0.015
	Steel	0.03	0.02	0.03
	Damage	Extensive	Extensive	Extensive

Note: ε_y =0.002; ε_{cu} =0.019

Table 4 – Maximum strains of D2 from time-history analysis

Return period (years)	Material Damage	Earthquake record number		
		1	2	3
475	Concrete	0.003	0.003	0.003
	Steel	0.004	0.005	0.004
	Damage	Repairable	Repairable	Repairable
975	Concrete	0.004	0.004	0.005
	Steel	0.006	0.006	0.008
	Damage	Repairable	Repairable	Repairable
2475	Concrete	0.007	0.006	0.007
	Steel	0.013	0.010	0.012
	Damage	Repairable	Repairable	Repairable

Note: ε_y =0.002; ε_{cu} =0.019

Table 5 – Maximum strains of D3 from time-history analysis

Return period (years)	Material Damage	Earthquake record number		
		1	2	3
475	Concrete	0.001	0.001	0.001
	Steel	0.0015	0.002	0.0017
	Damage	Minimal	Minimal	Minimal
975	Concrete	0.001	0.001	0.002
	Steel	0.002	0.002	0.002
	Damage	Minimal	Minimal	Minimal
2475	Concrete	0.003	0.001	0.002
	Steel	0.004	0.002	0.003
	Damage	Repairable	Minimal	Repairable

Note: ε_y =0.002; ε_{cu} =0.019

Table 6 – Damage states of D1, D2 and D3

Returen period (years)	475	975	2475
D1	Repairable	Repairable	Extensive
D2	Repairable	Repairable	Repairable
D3	Minimal	Minimal	Repairable

From Table 3, it can be seen that D1 tends to induce too much damage although life safety is protected. This will result in a very high repair cost. D3 tends to be too conservative with a huge amount of residual capacity. Considering the reinforcement ratio, proper construction may be very difficult. D2 is a design between D1 and D3, which may be the optimum choice in this study. For the design of regular Major-Route Bridges, since the CHBDC 2014 allows both PBD and FBD, it would be of interest to know what performance a FBD bridge can achieve. Based on the findings from the time-history analyses, a table of performance is presented based on the maximum strains of the FBD bridge (Table 7).

Table 7 – Performance of a bridge designed as per FBD in CHBDC 2014 (D2)

Return period (years)	Criteria

475	Concrete compressive strains (ε_c) ≤ 0.003 Steel strains (ε_{st}) ≤0.005
975	Concrete compressive strains (ε_c) ≤ 0.005 Steel strains (ε_{st}) ≤0.008
2475	Concrete compressive strains (ε_c) ≤ 0.008 Steel strains (ε_{st}) ≤0.014

4. Summary and Conclusions

This paper presents design examples of typical highway bridges based on CHBDC. The bridge was designed by using FBD as per the CHBDC 2006 (denoted as D1) and the CHBDC 2014 (denoted as D2), and also designed by PBD as per the CHBDC 2014 (denoted as D3). Site specific spectral accelerations and soil conditions p-y curves were used in the design. D2 had a higher reinforcement ratio than D1 because the CHBDC 2014 is meant to improve structrual safety. D3 had a much higher reinforcement ratio due to the strict requirements at 1/475-year event design. The 1/475-year event dominated the PBD.

The three designs (D1, D2 and D3) were assessed by performing time-history analyses. It was found that D1 and D2 fail to meet the criteria at 1/475. However, although D1 and D2 both met the criteria at 1/975 and 1/2475-year event, D2 showed much less damages than D1. By comparing bridge performances based on damage states, it might be confusing and inaccurate. Because the steel strain value can range from 0.002 to 0.015 for repariable damage, this may represent very different repair cost. D3 met all design criteria from CHBDC 2014. But it requires a very high reinforcement ratio and may pose some challenges and problems during construction. At the end, this paper presented the performance that a bridge designed as per FBD in CHBDC 2014 can achieve. The following conclusions are made from this study.

- Bridges designed as per FBD from CHBDC 2006 and CHBDC 2014 are able to protect life safety.
- CHBDC 2014 has very high requirements on seismic design compared to that of CHBDC 2006.
- PBD in CHBDC 2014 is very conservative comparing with other design approaches, which may significantly increase construction cost. Designing a bridge that remains essentially elastic in 1/475-year event is able to limit earthquake damages. However, avoidance of any yielding may be difficult. Hence, such strict seismic criteria will eventually lead designers/owners towards self-centering systems (e.g. shape memory alloy reinforced concrete piers, post-tensioning bridge piers etc).
- When performing PBD at different earthquake events, it is critical that the soil stiffness degradation be considered. This affects displacement demands significantly.

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