Canadian Journal of Civil Engineering
Revue canadienne de génie civil

Seismic performance comparison between force-based and performance-based design as per Canadian Highway Bridge Design Code (CHBDC) 2014

Journal:	<i>Canadian Journal of Civil Engineering</i>
Manuscript ID	cjce-2015-0419.R1
Manuscript Type:	Article
Date Submitted by the Author:	13-Apr-2016
Complete List of Authors:	Zhang, Qi; The University of British Columbia, School of Engineering Alam, M. Shahria; The University of British Columbia, Khan, Saqib; MMM Group Limited, Transportation Division Jiang, Jianping; MMM Group Limited, Transportation Division
Keyword:	response-earthquake load < Struct.Eng. & Constr.Mate, bridges < Struct.Eng. & Constr.Mate

SCHOLARONE™
Manuscripts

Seismic performance comparison between force-based and performance-based design as per Canadian Highway Bridge Design Code (CHBDC) 2014

School of Engineering, the University of British Columbia, Kelowna, BC, Canada V1V1V7

Qi Zhang

*MASc Graduate,
School of Engineering,
The University of British Columbia,
1137 Alumni Avenue, EME 3211,
Kelowna, BC, V1V1V7.
E-mail: qi_zhang@alumni.ubc.ca*

M. Shahria Alam^{*}

*Associate Professor, PEng.
School of Engineering
The University of British Columbia
1137 Alumni Avenue, EME 4225
Kelowna, BC, V1V1V7
TEL: (250) 807-9397 | Fax: (250) 807-9850
***Corresponding Author**
Email: shahria.alam@ubc.ca*

Saqib Khan

*Senior Project Engineer,
MMM Group Limited,
1045 Howe Street,
Vancouver, BC, V6Z2A9.
E-mail: KhanS2@mmm.ca*

Jianping Jiang

*Vice President,
MMM Group Limited,
1045 Howe Street,
Vancouver, BC, V6Z2A9.
E-mail: JiangJ@mmm.ca*

Word count: 9,100

Seismic performance comparison between force-based and performance-based design as per Canadian Highway Bridge Design Code (CHBDC) 2014

Qi Zhang; M. Shahria Alam^{*}; Saqib Khan and Jianping Jiang

Abstract: Performance-Based Design (PBD) has been first introduced in Canadian Highway Bridge Design Code (CHBDC) in 2014. PBD is the design that meets multiple performance criteria under different earthquake hazards. To investigate the impact of changes in CHBDC 2014, a four-span concrete highway bridge is designed and evaluated using Force-Based Design (FBD) and PBD methods as per CHBDC 2014, and FBD method as per CHBDC 2006. By incorporating soil-structure interaction (using p-y curves) nonlinear pushover and dynamic time-history analyses are conducted to assess the seismic performance of these bridges. Maximum strains of concrete and reinforcing-steel are compared among the three designs to determine their performance levels. It is concluded that PBD (CHBDC 2014) is highly conservative compared to both FBD (CHBDC 2014 and 2006). For the three-level PBD approach, the design is governed by the criterion of reinforcing-steel not yielding under the design earthquake (1 in 475 year return period).

Keywords: Seismic design; Concrete structures; Soil-structure interactions; Performance based design; Bridges; Codes.

^{*}Corresponding Author, Email: shahria.alam@ubc.ca Tel: +1.250.807.9397, Fax: +1.250.807.9850

Introduction

Bridges are critical civil infrastructures for the transportation network of any country. Hence, careful considerations are required for its proper design particularly in seismic regions. Conventionally, bridges are designed using Force-Based Design (FBD) method where it calculates seismic force demand by either single-mode or multi-mode spectral method. However, FBD has several limitations such as the inefficiency in connecting design process with target seismic performance. Detailed discussions on the limitations can be found in Priestley et al. (2007). In a recent study, Sheikh and Légeron (2014) reported that following the coded design rules does not automatically satisfy the descriptive performance goals in CHBDC 2006 (CSA 2006). Also, in several earthquakes, for example, 1994 Northridge Earthquake, even though structures designed based on FBD approach achieved the goal of protecting life safety, the costs of repair were unexpectedly high (Ghobarah 2001). To enhance FBD, Performance Based Design (PBD) has been developed by many researchers (Priestley 2000; Moehle and Deierlein 2004; Marsh and Stringer 2013; Billah and Alam 2016). PBD allows project owners to select target performance levels explicitly, which are to be used by design engineers. The performance levels can be determined based on damage states such as lateral drifts (overall or residual) and strains.

Under the PBD framework, structures can be designed based on any methods including FBD and displacement based design (DBD). DBD method for the bridge was proposed and improved by a number of researchers (Moehle 1992; Fajfar 1999; Chopra and Goel 2001; Kowalsky 2002; Dwairi and Kowalsky 2006). However, DBD is usually restricted to structures for which their deformed shape is easily estimated (Sullivan et al. 2003). Since CHBDC 2014 has not included DBD method, this study is focused on FBD and PBD.

In the current CHBDC 2014 (CSA 2014), both FBD and PBD are permitted to be used for the design of regular Major-Route Bridge. Major-Route Bridge is defined as the bridge that is a crucial part of the regional transportation and is critical to post-disaster event and security (CSA 2014). However, in the code, there is no indication if the two methods yield similar design results. This study investigates which design method is more conservative based on the case studies. Meanwhile, this study compares the design results from the new and old versions of the CHBDC (CSA 2006; CSA 2014).

Performance Based Design Criteria

In PBD, the performance criteria can be based on any qualitative and quantitative response parameters such as strains and drifts against prescribed probability demand levels (Ghobarah 2001). In CHBDC 2014 (CSA 2014), for Major Route Bridges the lower level design has a return period of 475 years and the upper design level has a return period of 2475 years. At the lower design level, the damage level is minimal damage (no yielding). At the upper design level, extensive damage is permitted but the reinforcing steel strain shall not exceed 0.005 and the core concrete shall not crush. Damage states from CHBDC 2014 are briefly presented in Table 1. Additionally, the CHBDC 2014 (CSA 2014) also has requirements on the level of analysis. For a Major Route Bridge, elastic static analysis is adequate in low seismicity regions but elastic dynamic analysis is required in high seismicity regions. The elastic dynamic analysis can be response spectral analysis or elastic time-history analysis.

The PBD philosophy is also adopted by a number of Departments of Transportation in U.S. At the lower design level, the structure should remain functional (serviceability) and at the higher design level the structure should avoid collapse. Although in the AASHTO LRFD Specifications

(AASHTO 2014) and the AASHTO LRFD Guide Specifications for Seismic Design (AASHTO 2013), only a single level design based on 1000-year return period is required, owners may decide to use other return periods for functionality design, such as 100-year return period (FHWA-NHI 2014). For example, the Oregon Department of Transportation (ODOT) requires two-level design (500 and 1000-year return period) for all new bridges (ODOT 2015). The South Carolina Department of Transportation (SCDOT 2008) also requires two-level designs which have 462-year and 975-year return period.

Force-Based Design and Performance-Based Design Process

The flowchart of FBD incorporating p-y method is shown in Figure 1. In step 1, the number and size of columns may be determined to maintain a certain amount of column axial load ratio (e.g. 10%). In step 2, cracked stiffness is used to incorporate stiffness reduction. The cracked stiffness ratio is estimated based on column cross section, axial load and reinforcement ratio. It can be found from a moment-curvature analysis. In step 3, p-y curves are incorporated into the modeling for soil-structure analysis. In step 4, the periods may be calculated from stiffness and mass of the structures by Equation 1

$$T=2\pi\sqrt{\frac{m}{k}} \quad \text{Equation 1}$$

where m is the effective mass and k is the stiffness. It should be noted that the soil effective stiffness changes with the change of lateral load after soil yields. From elastic stiffness to plastic stiffness range, soil loses its stiffness. Therefore, structures have different fundamental periods under different seismic loadings due to this reason.

Steps 4 to 6 are performed as an iterative response spectrum analysis process. Response spectrum analysis is a linear analysis where only linear soil spring can be used. Since soil loses its stiffness with the increase in lateral load after yielding, effective secant stiffness is used in the modeling. The secant spring stiffness can be determined by conducting modal analysis and response spectrum analysis iteratively. The iteration process is briefly explained in Figure 2 and Figure 3, where Figure 2 represents the global response of the structure and Figure 3 represents the local response of the soil spring. At the beginning of the iteration, stiffness and displacement of the spring can be assumed as K_1 and Y_1 . Then response spectrum analysis is performed, which generates another displacement Y_2 . If Y_2 is different from Y_1 , then Y_2 should be used in the local spring model to find out the corresponding stiffness K_2 . The iteration should be continued until the stiffness and displacement from the global structure model converge with the local spring model. In step 7, ductility factor is defined by design codes and used to reduce elastic force demand. In step 8, base shear is distributed to columns according to their stiffness. Step 9 and step 10 are performed at the end to make sure the ductile structure can achieve the desired plastic mechanism.

In the CHBDC 2014 (CSA 2014) the major difference between PBD and FBD is that PBD requires more complicated and refined analysis such as response spectral analysis and elastic time-history analysis. The preliminary design results may be based on any design methods but the final design has to be checked using required methods. A flowchart of PBD is shown in Figure 4.

Case Study

A four-span concrete bridge with multi-column bents is selected for the case study. This bridge consists of a 235 mm thick concrete deck and 12 lines of 1.73m deep precast I girders. The total length of the bridge is 100 meters and the width of the bridge is 40 meters. The bridge has three pier bents and two abutment bents. Each of the bent has eight columns. The net height of each column is about 6 meters and the length of each pile is about 20 meters. The pier caps are made integral at Pier 1, Pier 2 and Pier 3. The general arrangement of the bridge is shown in Figure 5. From Abutment 0 to Abutment 4, the span lengths are 14 meters, 34 meters, 36 meters and 14 meters.

In the design phase, the bridge model is built in SAP2000 (CSI 2010). The soil-structure interaction is simulated using a series of p-y springs. Soil-structure interaction is an important factor that affects the seismic performance of the bridge (Dash et al. 2008). Figure 6 shows a typical p-y curve where the soil loses its strength and stiffness with the increase of displacement. In p-y curves, p stands for lateral resistance force per unit pile length from soil, and y stands for lateral displacement of piles. The finite element model of the bridge is shown in Figure 7 and the site-specific response spectra are shown in Figure 8.

In this research, two separate FBDs are conducted as per CHBDC 2006 (CSA 2006) and the CHBDC 2014 (CSA 2014), respectively, which are denoted as D1 and D2. The PBD is conducted as per the CHBDC 2014 (CSA 2014), which is denoted as D3. D1 is designed for an earthquake return period of 475 years as per CHBDC 2006 (CSA 2006) and D2 is designed for a return period of 2475 years CHBDC 2014 (CSA 2014). D3 is designed for three levels with 475 years, 975 years and 2475 years return periods as per CHBDC 2014 (CSA 2014). The design

results and corresponding hazard return period are listed in Table 2. The column cross sections of the three designs are shown in Figure 9. It should be noted that although the structural member stiffness of D1 and D2 are generally the same, their overall structural stiffness is different because of the difference in soil springs. D2 is designed for a higher seismic demand, which results in softer soil springs and longer fundamental periods. Because of the difference in soil stiffness, there is difference in fundamental periods.

Comparing the two FBDs, D2 has a slightly higher reinforcement ratio due to the higher seismic demand. D3 shows a significant increase in reinforcement ratio and column sectional area compared to D1 and D2. This is due to the requirement that reinforcing steel strains shall not exceed yield at 1 in 475-year earthquake event (CSA 2014). The transverse reinforcement spacings of three different designs are 75 mm, 75 mm and 65 mm for D1, D2 and D3, where the confinement factors are 1.40, 1.49 and 1.46, respectively. The lateral reinforcement is provided such that shear failure does not occur and flexural behavior with enough ductility is guaranteed.

Performance Assessment Based On Pushover Analysis

To assess the performance of the bridge, pushover analysis is conducted in the transverse direction of the bent. The bents were pushed to the displacement demands calculated from response spectrum analysis. Pushover analysis was performed using SeismoStruct (SeismoSoft 2010a). SeismoStruct (SeismoSoft 2010a) is a fibre based finite element program that has excellent capability in non-linear analysis. Performance criteria such as strains can be directly shown in SeismoStruct (SeismoSoft 2010a). The concrete confinement model by Mander et al. (1988) and Menegotto-Pinto steel model (Menegotto and Pinto 1973) are used.

As shown in Table 2, D1 is designed as per CHBDC 2006 and its reinforcement ratio is 1.9%. The seismic design performance criteria from the CHBDC 2014 are used to assess the seismic

performance of D1. Figure 10 to Figure 14 show the pushover curves with displacement demands and strain limits. The displacement demands from different events are shown with dashed vertical lines. Strain criteria are marked on the curves. As shown from Figure 10 to Figure 14, all the bents reach first yielding of longitudinal reinforcement under the displacement demand of 1 in 475-year event. Generally, the first yielding happens when bents reach about half of the displacement demands at 1 in 475-year event, which means that none of the bents meet the performance criteria in the CHBDC 2014 (CSA 2014). For 1 in 975-year event, CHBDC 2014 requires that reinforcing steel strains shall not exceed 0.015. It is observed that Abutment 4 barely meets this requirement thus the bridge may reach extensive damage states at 1 in 975 years event. Abutment 4 shows damage much earlier than the other bents. This can be explained by the specific site conditions. The bridge site is defined as Site Class F, which is composed of very soft soils. Site-specific p-y curves are developed for each pier. Pier 3 and Abutment 4 are founded on similar soils which are softer than the other three bents. Therefore, Pier 3 and Abutment 4 have less lateral stiffness and higher displacement demands compared to other bents. It is also found that all the bents meet the criteria at 1 in 2475 year event since the reinforcing steel strain of 0.05 is not reached. The pushover analysis shows that the bridge designed as per CHBDC 2006 is able to protect life safety for considered earthquake events.

When performing PBD, nonlinear pushover or time-history analysis is required at the design phase. In the PBD of this study, it is realized that Abutment 4 experiences the highest displacement demand and shows the most damage, so that pushover analysis is carried out only on Abutment 4. The pushover curve of Abutment 4 is shown in Figure 15. The CHBDC 2014 requires that reinforcing steel strains shall not exceed yield at 1 in 475-year event. This requirement results in a very high longitudinal reinforcement ratio in piers, which is 5.3%. When

the bent is pushed to the displacement demand, the maximum reinforcing steel strain is 0.0024, which is considered meeting the requirement of the CHBDC 2014. At 1 in 975-year event, the concrete strain is still smaller than 0.004, which is in minimal damage level. The reinforcing steel strain only increases to 0.01 after 1 in 2475-year event, and concrete strain is smaller than 0.006. The reinforcing steel strain of 0.015 is not reached throughout the analysis. It is also confirmed that when the performance criteria of 1 in 475 year are satisfied, the bridge may not even experience repairable damage level at maximum considered earthquake level. Therefore, D3 shows the most conservative results in comparison with D1 and D2.

Performance Assessment Based On Time-History Analysis

To conduct a rigorous assessment of the seismic performance of three designs, time-history analyses are carried out. In the time-history analysis, seven earthquake records are selected from the Canadian Association for Earthquake Engineering (Naumoski et al. 1988). The records are scaled based on site-specific response spectra using the program SeismoMatch (SeismoSoft 2010b). Acceleration loads are applied in both longitudinal and transverse directions. Table 3 lists the records for time-history analysis. Two original acceleration time histories are plotted in Figure 16a. The scaled acceleration time histories are plotted in Figure 16b. Figures 17a and 17b show the unmatched and matched accelerogram spectra with the target spectra, respectively.

Maximum strains from the time-history analyses are presented in Tables 4 to 6 for three designs (D1, D2 and D3). Tables 4 to 6 show results from the three records and Table 7 shows the damage states determined from average strains in all records. From the time-history analysis, it is concluded that D1 fails to meet the performance criteria at 1 in 475-year event in CHBDC (CSA 2014). This conclusion is the same with the findings from pushover analysis. D2 also fails to meet the performance criteria at 1 in 475-year event in CHBDC (CSA 2014). D3 meets the

performance criteria at all earthquake events and only reaches repairable damage states at 1 in 2475-year event. Due to extremely conservative design, there is a huge amount of redundancy in terms of residual capacity after the occurrence of the first rebar yielding in D3. The ductility after first yielding cannot be well utilized when such conservatism is incorporated. As shown in Figure 18, for a ductile structure design, it is expected that the plastic hinge first forms on the top of columns. Then the hinge forms on the piles, and lastly at the bottom of columns.

Comparing the three designs, D1 tends to induce a lot of damage, although life safety is protected. This may result in a high repair cost after earthquakes. D3 tends to be too conservative with a huge amount of residual capacity. The longitudinal reinforcement ratio of D2 is in between D1 and D3. A table of performance criteria that D2 achieves is presented in Table 8.

Summary and Conclusions

Examples of typical highway bridge designs are presented in this paper. The bridge is designed using FBD as per the CHBDC 2006 (denoted as D1), FBD as per 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 are considered in the design. The soil structure interactions are incorporated by using a series of p-y curves. D2 has a higher reinforcement ratio than D1 as the design hazard return period in CHBDC 2014 has increased. D3 has a much higher reinforcement ratio compared to D1 and D2 due to the strict requirements at 1 in 475-year event design.

After designing the bridge with three different code requirements, pushover analysis and time-history analysis are conducted to evaluate their seismic performance. The results from pushover analyses and time-history analyses are similar in terms of damage states. It is found that D1 and D2 fail to meet the performance criteria in CHBDC 2014 (CSA 2014) at 1 in 475 year event. However, although D1 and D2 both meet the criteria at 1 in 975 and 1 in 2475-year event, D2

shows less damage than D1. D3 meets the criteria at all earthquake events. However, D3 requires a very high reinforcement ratio. At the end, this paper presents a table listing the performance of a bridge designed as per FBD in CHBDC 2014. To summarize, the following conclusions are made from this study.

- Bridges designed as per CHBDC 2006 and CHBDC 2014 can provide desirable seismic performance in terms of life safety.
- Comparing with FBD in CHBDC 2006, FBD in CHBDC 2014 sets higher requirements for seismic loading. The increase in return period from 475 years to 2475 years results in a reinforcement ratio increase of 0.8% in this case study.
- The PBD in CHBDC 2014 is more conservative in comparison to FBD in CHBDC 2006 and CHBDC 2014. Although this will limit earthquake damage, it may be challenging in terms of construction (5.3% reinforcement ratio – leading to congestion). It is likely that for Major Route Bridges, the design will be governed by the lower design level with the requirement of no reinforcing steel yielding at 1 in 475-year earthquake event.
- When performing PBD at different earthquake events, it is critical that the soil stiffness degradation is considered. The soil-structure interaction affects displacement demands significantly.

Acknowledgments

The work presented here was carried out under the support from MMM Group Limited as part of The Natural Sciences and Engineering Research Council of Canada (NSERC) – Industrial Postgraduate Scholarships Program. The financial supports provided by MMM Group Limited and NSERC are gratefully acknowledged.

References

- AASHTO. 2013. AASHTO Guide Specifications for LRFD Seismic Bridge Design. American Association of State Highway and Transportation Officials.
- AASHTO. 2014. AASHTO LRFD Bridge Design Specifications. American Association of State Highway and Transportation Officials.
- Billah, AHMM, & Alam, MS. 2016. Performance (residual drift)-based seismic design of Shape Memory Alloy (SMA) reinforced concrete bridge pier - Part 2: Methodology and Design Example. *Journal of Structural Engineering*, ASCE, manuscript no. STENG-4012.
- Chopra, A.K., and Goel, R.K. 2001. Direct displacement-based design: use of inelastic vs. elastic design spectra. *Earthquake Spectra* 17(1): 47-64.
- CSA. 2006. Canadian highway bridge design code. Canadian Standards Association.
- CSA. 2014. Canadian highway bridge design code. Canadian Standards Association.
- CSI. 2010. Integrated Finite Element Analysis and Design of Structures Basic Analysis Reference Manual. Computers and Structures, Inc., Berkeley, California, USA.
- Dash, S.R., Bhattacharya, S., Blakeborough, A., and Hyodo, M. 2008. PY Curve to model lateral response of pile foundations in liquefied soils. 14th World Conference on Earthquake Engineering Beijing, China. October 12-17, 2008.
- Dwairi, H., and Kowalsky, M. 2006. Implementation of inelastic displacement patterns in direct displacement-based design of continuous bridge structures. *Earthquake Spectra* 22(3): 631-662.
- Fajfar, P. 1999. Capacity spectrum method based on inelastic demand spectra. *Earthquake Engineering & Structural Dynamics* 28(9): 979-993.

- FHWA-NHI. 2014. LRFD Seismic Analysis and Design of Bridges Reference Manual. U.S. Department of Transportation Federal Highway Administration, Publication No. FHWA-NHI-15-004.
- Ghobarah, A. 2001. Performance-based design in earthquake engineering: state of development. *Engineering structures* 23(8): 878-884.
- Kowalsky, M.J. 2002. A displacement - based approach for the seismic design of continuous concrete bridges. *Earthquake engineering & structural dynamics* 31(3): 719-747.
- Mander, J.B., Priestley, M.J., and Park, R. 1988. Theoretical stress-strain model for confined concrete. *Journal of structural engineering* 114(8): 1804-1826.
- Marsh, M.L., and Stringer, S.J. 2013. Performance-based Seismic Bridge Design. Transportation Research Board, Washington, D.C.
- Menegotto, M., and Pinto, P. 1973. Method of analysis for cyclically loaded reinforced concrete frames including changes in geometry and non-elastic behavior of elements under combined normal forces and bending moment. IASBE Proceedings.
- Moehle, J. 1992. Displacement-based design of RC structures subjected to earthquakes. *Earthquake spectra* 8(3): 403-428.
- Moehle, J., and Deierlein, G.G. 2004. A framework methodology for performance-based earthquake engineering. 13th World Conference on Earthquake Engineering Vancouver, B.C., Canada. August 1-6, 2004.
- Naumoski, N., Heidebrecht, A., and Tso, W. 1988. A selection of representative strong motion earthquake records having different A/V ratios. McMaster University, Earthquake Engineering Research Group.

ODOT. 2015. Bridge Design and Drafting Manual. Oregon Department of Transportation. Salem, OR.

Priestley, M. 2000. Performance based seismic design. Bulletin of the New Zealand Society for Earthquake Engineering 33(3): 325-346.

Priestley, M., Calvi, G., and Kowalsky, M. 2007. Direct displacement-based seismic design of structures.

SCDOT. 2008. SCDOT seismic design specifications for highway bridges. Columbia, SC.

SeismoSoft, S. 2010a. A computer program (SeismoStruct) for static and dynamic nonlinear analysis of framed structures.

SeismoSoft, S. 2010b. A software (SeismoSignal) capable of adjusting earthquake accelerograms.

Sheikh, M.N., and Légeron, F. 2014. Performance based seismic assessment of bridges designed according to Canadian Highway Bridge Design Code. Canadian Journal of Civil Engineering 41(9): 777-787.

Sullivan, T., Calvi, G., Priestley, M., and Kowalsky, M. 2003. The limitations and performances of different displacement based design methods. Journal of Earthquake Engineering 7(spec01): 201-241.

Table 1. Performance Criteria (CHBDC, 2014)

Table 2. Design cases

Table 3. Earthquake records (Naumoski et al., 1988)

Table 4. Maximum strains of D1 from time-history analysis

Table 5. Maximum strains of D2 from time-history analysis

Table 6. Maximum strains of D3 from time-history analysis

Table 7. Damage states of D1, D2 and D3

Table 8. Performance of a bridge designed as per FBD in CHBDC 2014 (D2)

Draft

Figure 1 Force-Based Design incorporating p-y method

Figure 2 Global response of structure

Figure 3 Local response of soil spring

Figure 4 Performance-Based Design

Figure 5 General arrangement (reproduced with permission from the project team)

Figure 6 Typical p-y curve

Figure 7 Finite element model in SAP2000

Figure 8 Response spectra

Figure 9 Column section

Figure 10 Abutment 0 pushover curve in D1

Figure 11 Bent 1 pushover curve in D1

Figure 12 Bent 2 pushover curve in D1

Figure 13 Bent 3 pushover curve in D1

Figure 14 Abutment 4 pushover curve in D1

Figure 15 Abutment 4 Pushover curve in D3

Figure 16a Original acceleration time histories

Figure 16b Scaled acceleration time histories

Figure 17a Target spectra with original spectra

Figure 17b Target spectra with matched spectra

Figure 18 Ductile bent yielding sequence

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.

Draft

Table 2. Design cases

Case no	Design method	Design Code CHBDC	Column diameter (m)	Pier 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

Draft

Table 3. Earthquake records (Naumoski et al. 1988)

Record Number	Earthquake	Date	Magnitude	Site	Max. Acc. A(g)	Max. Vel. V(m/s)
1	Imperial Valley California	1940-5-18	6.6	El Centro	0.348	0.334
2	Kern County California	1952-7-21	7.6	Taft Lincoln School Tunnel	0.179	0.177
3	San Fernando California	1971-2-9	6.4	Hollywood Storage P.E. Lot, L.A.	0.211	0.211
4	San Fernando California	1971-2-9	6.4	Griffith Park Observatory, L.A	0.18	0.205
5	San Fernando California	1971-2-9	6.4	234 Figueroa St., L.A.	0.199	0.167
6	Near East Coast of Honshu, Japan	1971-8-2	7	Kushiro Central Wharf	0.078	0.068
7	Monte Negro Yugoslavia	1979-4-15	7	Albatros Hotel, Ulcinj	0.171	0.194

Table 4. 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: $\epsilon_y=0.002$; $\epsilon_{cu}=0.019$

Draft

Table 5. 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: $\epsilon_y = 0.002$; $\epsilon_{cu} = 0.019$

Draft

Table 6. 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: $\epsilon_y=0.002$; $\epsilon_{cu}=0.019$

Draft

Table 7. Damage states of D1, D2 and D3

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

Draft

Table 8. Performance of a bridge designed as per FBD in CHBDC 2014 (D2)

Return period (years)	Criteria
475	Concrete compressive strains (ϵ_c) $\leq$ 0.003 Steel strains (ϵ_{st}) $\leq$ 0.005
975	Concrete compressive strains (ϵ_c) $\leq$ 0.005 Steel strains (ϵ_{st}) $\leq$ 0.008
2475	Concrete compressive strains (ϵ_c) $\leq$ 0.008 Steel strains (ϵ_{st}) $\leq$ 0.014

Draft

Figure 1 Force-Based Design incorporating p-y method
114x92mm (199 x 199 DPI)

Figure 2 Global response of structure
137x111mm (96 x 96 DPI)

Figure 3 Local response of soil spring
76x64mm (147 x 147 DPI)

Figure 4 Performance-Based Design
135x57mm (246 x 246 DPI)

Draft

Figure 5 General arrangement (reproduced with permission from the project team)
180x48mm (183 x 183 DPI)

Figure 6 Typical p-y curve
68x53mm (135 x 135 DPI)

Figure 7 Finite element model in SAP2000
117x75mm (199 x 199 DPI)

Figure 8 Response spectra
78x59mm (193 x 193 DPI)

Figure 9 Column section
130x33mm (199 x 199 DPI)

Draft

Figure 10 Abutment 0 pushover curve in D1
134x93mm (96 x 96 DPI)

Figure 11 Bent 1 pushover curve in D1
127x98mm (96 x 96 DPI)

Figure 12 Bent 2 pushover curve in D1
128x101mm (96 x 96 DPI)

Figure 13 Bent 3 pushover curve in D1
133x88mm (96 x 96 DPI)

Figure 14 Abutment 4 pushover curve in D1
133x91mm (96 x 96 DPI)

Figure 15 Abutment 4 Pushover curve in D3
129x92mm (96 x 96 DPI)

Figure 16a Original acceleration time histories
107x62mm (155 x 155 DPI)

Figure 16b Scaled acceleration time histories
152x96mm (96 x 96 DPI)

Figure 17a Target spectra with original spectra
200x118mm (96 x 96 DPI)

Figure 17b Target spectra with matched spectra
198x117mm (96 x 96 DPI)

Figure 18 Ductile bent yielding sequence
128x38mm (219 x 219 DPI)

Draft