Parametric Study to Estimate Seismic Displacement Demands of Balanced Cantilever Bridges in Service and Construction Conditions

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Abstract

Balanced cantilever box-girder bridges may be more seismically vulnerable during construction than after completion due to the higher flexibility and lower redundancy of the uncompleted structure. To estimate the seismic response of these bridges during construction, the Colombian bridge design Code defines a spectral linear-elastic, force-based approach considering a seismic hazard level lower than that of the in-service condition, along with the same viscous damping ratio of design equal to 5%. However, as these bridges present different stiffnesses in both conditions (construction and in-service), lower spectral acceleration demand and associated forces that are calculated for construction may give an erroneous idea that the bridge is less seismically vulnerable during construction than in-service. For this reason, this study aims to investigate the effect of the following parameters on the spectral displacement and acceleration demands during construction and in-service of balanced cantilever box-girder bridges: damping ratio, length and number of spans (two, three, four, and five), column height, and level of progress of the cantilever construction before building the closure segment. The bridges were analyzed for the longitudinal direction only, as single-degree-of-freedom systems. The results show that the design acceleration demand is larger for all in-service bridges, as expected due to the increase in stiffness after completion. However, balanced cantilever box-girder bridges can be subjected to equal or greater seismic displacement demands during construction than those in-service condition.

Keywords: *balanced cantilever bridge; during construction; seismic vulnerability, displacement spectra; seismic demand.*

1. Introduction

Prestressed concrete box-girder bridges built by the balanced cantilever method are widely used in Colombia because that construction method is advantageous given the topographical characteristics of this country. During this construction process, bridges have less stiffness and less redundancy than after the structure is completed (i.e., in service condition), which implies differences in the seismic response between both conditions.

Some numerical studies have been conducted on the seismic vulnerability of balanced cantilever bridges. Chang et al. (2004) analyzed a two-span cable-stayed bridge that was struck by the 1999 Chi-Chi earthquake during construction. The authors found that the bridge behaved unsymmetrically during the earthquake because one girder end was supported on the abutment while the other was free. The asymmetric response caused moderate damage to the columns and major damage in the girder due to large impact forces between the girder and the abutments. Wilson and Holmes (2007) analyzed the seismic response of a cable-stayed bridge in service and during several construction stages through linear-dynamic procedures. The authors calculated the seismic forces in different structural elements and, through a probabilistic approach, determined that the probability of exceedance of the seismic forces in the partially completed bridge surpassed those calculated for the completed system. A similar conclusion was found by Benjumea and Chio Cho (2013) for an extradosed bridge, who followed the procedure developed by Wilson and Holmes (2007). These studies indicate that there may be a high degree of seismic vulnerability on balanced cantilever bridges associated to the stages before the closure segment is built.

Current codes and regulations for the design of bridges provide guidelines for the seismic analysis and design of bridges during construction. For example, the Colombian bridge design Code (CCP-14) (AIS 2014), which is based on the AASHTO LRFD bridge design specifications (*AASHTO* 2012) and (*AASHTO* 2014), states that bridges built by the balanced cantilever method shall be analyzed during construction by a linear-elastic force-based approach. In this approach, the seismic demand is calculated through a pseudo-acceleration design response spectrum that depends on the soil profile type and assumes a viscous damping ratio (ξ) equal to 5%, thus, the same viscous damping ratio used for service condition.

The issue with the force-based design method is that it only takes into account the seismic spectral pseudo-acceleration demand and ignores other seismic demand parameters like the velocity and displacement. Acceleration, velocity, and displacement depend on the natural period of vibration of bridges and the trend of their response spectra is different each other. Because of this, it is possible that when comparing two structures with different fundamental periods, higher acceleration demand may occur in the stiffer structure but higher displacement demand in the other. This implies that bridges during construction may be more prone to structural damage better correlated with displacement than with acceleration even if the acceleration results were higher in-service condition. Such correlation between damage and displacement may be seen from the moment-curvature diagram of a single cantilever reinforced concrete member that may well represent the column stiffness of the bridge during service condition. Also, the issue may be even greater because, as found by Hernandez et al. (2020) and Panetsos et al. (2008), the viscous damping ratio of balanced cantilever bridges is usually lower in construction than in-service and therefore the seismic displacement demand during construction may be currently underestimated.

Despite the issues mentioned above, most of the available literature on the seismic response and displacement demands on balanced cantilever bridges is associated to the in-service condition. For example, Bardakis and Fardis (2011) compared the displacement demands of completed box-girder bridges with different structural geometry calculated by linear modal spectral analysis and nonlinear dynamic analysis. Li et al. (2018) compared the seismic performance and displacement demand of a long-span box-girder bridge with tall columns when it is subjected to synchronic and asynchronous uniform ground motion. Xinle et al. (2007) and Bavraktar et al. (2020) studied the effects of vertical ground motions in the seismic response of box-girder bridges subjected to near-fault ground motions.

Based on the above, this research was proposed to study the effects of the span length, column height, viscous damping ratio, and soil type on the spectral pseudo-acceleration and displacement demand following CCP-14 guidelines for four balanced cantilever box-girder bridges during service and during three construction stages.

2. Methodology

2.1. Configuration of studied bridges

Figure 1 shows the configuration of the balanced cantilever box-girder bridges that were analyzed, four in-service configurations (S1 to S4) and three construction stages (C1, C2, and C3), that are in terms of the percentage of the cantilever built (30%, 70%, and 100%, respectively). In that figure, L is the main span length, H is the column height, and m' is the mass of each main span. Note that the side span lengths and masses are half of those in the main span, and the columns height are equal for a given bridge. Schematics of the cross sections of the girders and columns of the bridges are shown in Figure 2. The bridges consist of unicellular box-girder with height varying parabolically while the columns are hollow-rectangular and with constant cross section. These elements were sized using the equations listed in Table 1, which are based on the recommendations by Ariñez and Astiz (2012) and Manterola (2006) for balanced cantilever box-girder bridges. Parameters B, t, a, and A represent the deck width, column walls thickness, and the girder height at mid-span and over the columns, respectively. B and t were kept constant and equal to 11 m and 0.6 m, respectively, as these are typical values found in many Colombian bridges.

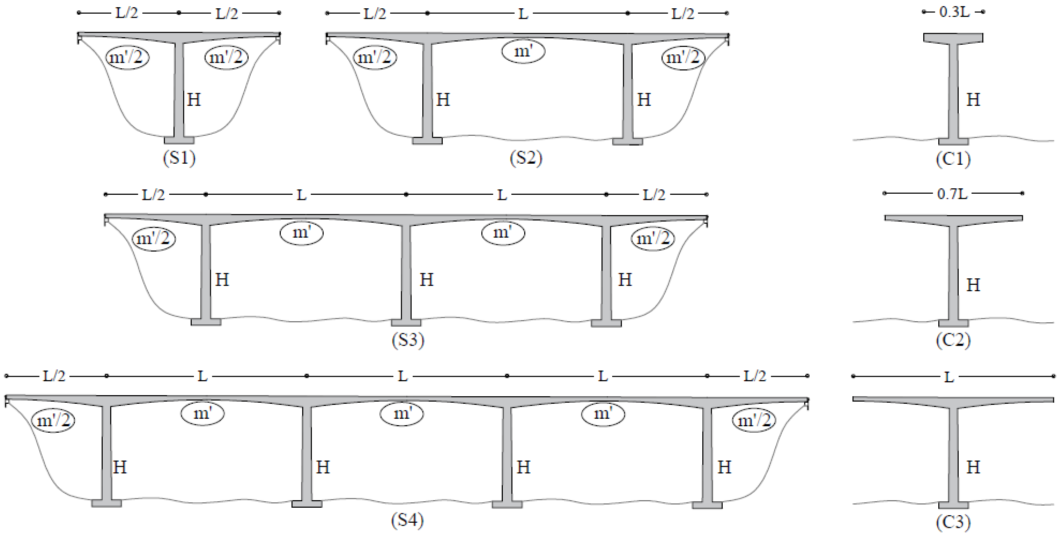

Figure 1. Structural configuration of studied bridges in service (S1, S2, S3, and S4) and during construction (C1, C2, and C3).

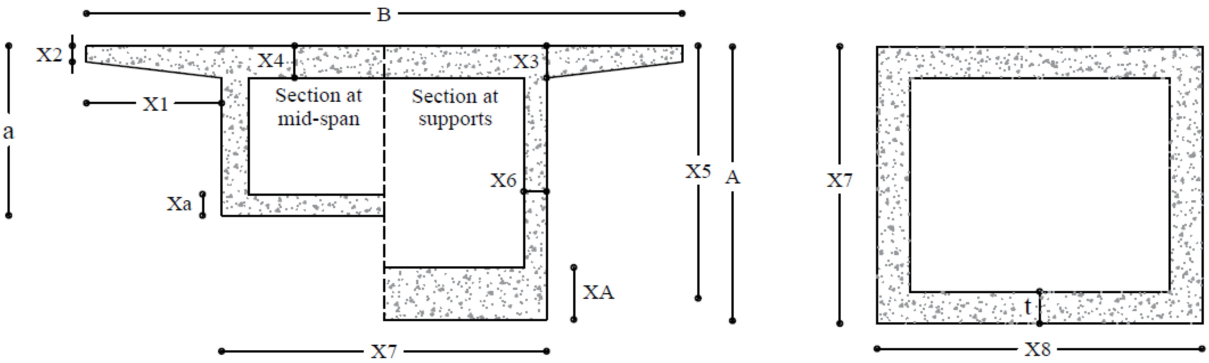

Figure 2. Dimensions of girders (left) and hollow columns (right).

Table 1. Equations to calculate the girder and columns dimensions.

Parameter (in m)	Equation	Parameter (in m)	Equation
a	$L/40$	X3	$(0.5/5.2)X7 + (1/300)$
A	$L/20$	X4	$(0.1/3.2)X7 + (3/40)$
Xa	0.2	X5	$A - X4$
XA	$(0.00015BL^2)/(AX7)$	X6	$(0.25/6)A + (1/12)$
X1	0.4X7	X7	$B/1.8$
X2	0.2	X8	$0.06H + 2$

2.2. Parameters

Span length, column height, soil type and viscous damping ratio were varied to determine their effect on the seismic spectral pseudo-acceleration and displacement demand on the bridge configurations shown in Figure 1. A total of 2,280 models in service and 1,710 models in construction were analyzed after varying those parameters, which are detailed below.

2.2.1. Span length (L) and column height (H)

The span length (L) and column height (H) were selected based on the results of dispersion statistical analysis from 28 actual balanced cantilever box-girder bridges built in Colombia. The information of those bridges was collected from structural drawings of the original designs. The results of the analysis are shown in Figure 3. The gray dots shown in that figure correspond to each bridge, while the continuous and dashed black lines represent the mean and the upper bound that defines whether the data is representative or not. Considering that all data of the statistical analysis are representative, the main span lengths (L) selected for this study were 60 m, 130 m, and 200 m, whereas the height of the columns (H) was varied from 10 m to 100 m, with increments of 5 m.

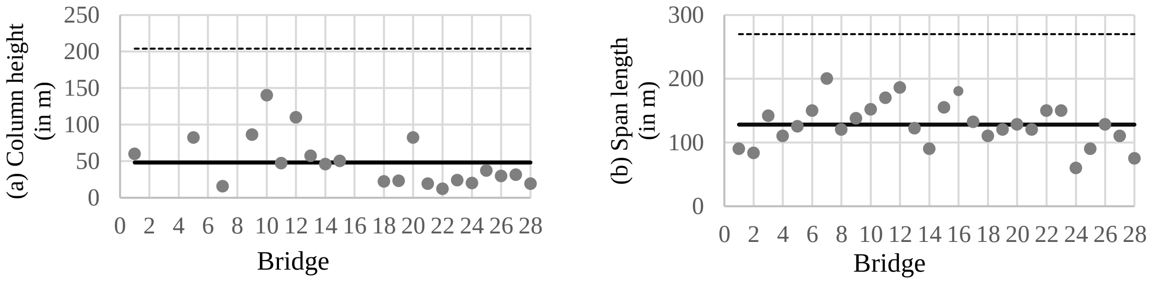

Figure 3. Statistical analysis of: (a) column heights and (b) main span lengths of 28 bridges.

2.2.2. Soil type

The soil rigidity modifies the amplitude of the design acceleration response spectrum. This parameter was varied in this study by assuming that the bridges are built on soil profile types A, B, C, D, and E. The average shear wave velocity (V_s) of each soil profile according to CCP-14 is shown in Table 2. The modification factors for the peak ground acceleration (PGA) and the constant acceleration (F_a) and velocity (F_v) regions of the design spectrum were determined by following Tables 3.10.3.2-2 and 3.10.3.2-3 in CCP-14.

Table 2. Soil type and shear wave velocity in CCP-14.

Soil type	**V_s (in m/s)**
A	$V_s \geq 1500$
B	$1500 > V_s \geq 760$
C	$360 > V_s \geq 760$
D	$180 > V_s \geq 360$
E	$V_s < 180$

2.2.3. Viscous damping ratio (ξ)

Table 3 shows typical viscous damping ratios (ξ) measured by different authors on balanced cantilever box-girder bridges in service and construction. Those damping ratios are associated to different mode shapes in the vertical, transverse, and longitudinal directions of the bridges. It can be seen that the damping ratios are lower than the standard 5% recommended by CCP-14 in most of the identified mode shapes, and the damping ratios of the uncompleted structures (during construction) are lower than those of the completed bridges (in-service). To study the effects of the viscous damping ratio on the seismic demand of the bridges considered in this investigation, two cases were analyzed. In Case 1, a 5% viscous damping ratio was used for the service and staged-construction conditions of the bridges. This case follows the recommendations in CCP-14. In Case 2, the viscous damping ratios used for construction and service conditions were 1.3% and 2%, respectively.

Table 3. Measured viscous damping ratios on balanced cantilever bridges during construction and in service.

Stage	**Mode**	**Damping ratios (%)**		
		(Hernandez et al. 2020)	(Panetsos et al. 2008)	(Ntotsios et al. 2009)
Construction	Vertical	0.68 – 1.14	0.25 – 0.43	Not measured
	Transverse	0.57 – 1.26	0.40 – 1.27	Not measured
	Longitudinal	0.63 – 1.32	0.18 – 0.42	Not measured
Service	Vertical	0.91 – 0.93	Not measured	0.60 – 0.70
	Transverse	0.90 – 1.90	Not measured	1.40 – 2.00
	Longitudinal	1.35 – 1.50	Not measured	1.80

2.3. Assessment of seismic demands

2.3.1. Structural idealization

The bridges were analyzed for the longitudinal direction only as single-degree-of-freedom systems (SDOF) after assuming that the superstructure is infinitely rigid. The total mass of the SDOF systems (*m*), both during construction and in-service, was calculated including the mass of each span of the superstructure (*m'* in Fig. 1) and thirty percent of the columns masses. The mass of each span was calculated based on its volume and by using a unit weight of 24 kN/m^3 for the concrete. This idealization allowed to approximate the period of vibration of the bridges by means of Equation (1). In that equation, the lateral stiffness (*K*) was calculated as the sum of the longitudinal bending stiffness of each column. The columns were assumed to behave in double and single curvature during service and construction, respectively; therefore, their stiffnesses were calculated as $3EI/H^3$ and $12EI/H^3$, respectively, where *I* is the cracked moment of inertia of the columns and *E* is their modulus of elasticity. The nominal compressive strength (f'_c) of the columns was 35 MPa since this is a typical design value for Colombian bridges. However, the modulus of elasticity (*E*) in Equation (2) was calculated using the expected (probable) concrete compressive strength (f'_{ce}) rather than the nominal value. The expected concrete strength was taken as $1.3f'_c$, as suggested in section 8.4.4 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design (*AASHTO* 2011). Cracking of the columns was taken into account by reducing the gross moment of inertia by a factor of 0.5, as recommended in CCP-14. Long-term effects were not included in the analyses, and the periods of vibrations were not modified by neither soil-structure interaction effects nor the viscous damping ratio.

$$T = 2\pi\sqrt{\frac{m}{K}} \tag{1}$$

$$E = 4700\sqrt{f'_{ce}} \tag{2}$$

2.3.2. Acceleration and displacement demands

The pseudo-acceleration (A_i) demands on the bridges for each soil type *i* were calculated using the design response spectrum determined by following section 3.10 of CCP-14. Figure 4a shows an example of that spectrum for the reference soil type B. The spectrum was calculated for the city of Bucaramanga, Colombia, which is located on a region with high seismic hazard. The pseudo-acceleration response spectrum for the completed bridge corresponds to a design earthquake with a probability of occurrence of 7% in 75 years (return period of 1033 years). The vertical line showed in Figure 4a corresponds to the short period limit of the spectrum (*Ts*). To consider the short period of exposure of balanced cantilever bridges during construction, CCP-14 establishes that the seismic hazard during construction must be calculated by dividing the response spectrum of the completed structure over a factor not greater than two. Accordingly, the pseudo-acceleration spectrum for the bridge models during construction considered in this study was calculated as half of the response spectrum of the completed bridges.

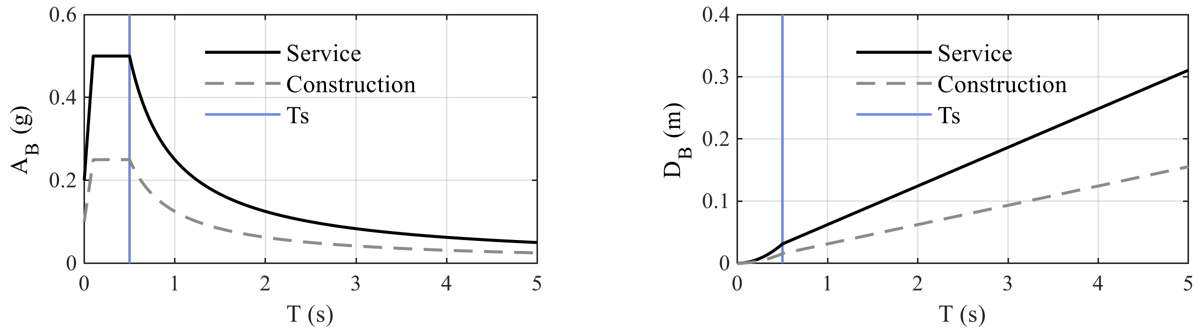

Figure 4. Examples of pseudo-acceleration and displacement spectra for the construction and service conditions.

Because CCP-14 does not include any procedure or recommendation to calculate the design spectral displacements, the displacement spectrum (Fig. 4b) was calculated by dividing the pseudo-acceleration spectral ordinates (A_i) over the squared angular frequency, as shown in Equation (3). The displacement spectrum built by this methodology implies a constant increase of the spectral displacement beyond the short period limit, which may not be realistic. The spectral acceleration and displacement demands were multiplied by the damping modification factors (DMF) shown in Equation (4) to account for the viscous damping ratio values of the bridge model. This factor is recommended in section 4.3.2 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design (*AASHTO* 2011). Note that the DMF is equal to one when ξ = 5% and increases for lower values of damping ratio. For this reason, the DMF calculated for ξ=1.3% is 19% higher than that calculated for ξ=2.0%; therefore, the ordinates of the pseudo-acceleration and displacement spectra for the bridges during construction in Case 2 are 40.6% lower than those calculated for the completed bridges.

To compare the seismic demands during construction and in-service conditions for a given bridge, the spectral pseudo-acceleration and displacement demands calculated for the former condition were normalized with respect to those of the completed bridge. The normalized ratios are shown in Equations (5) and (6). A value larger than one for ψ_A or ψ_D indicates that the seismic spectral demand is greater during construction than in-service condition.

$$D_i = \frac{A_i}{\left(\frac{T_i}{2\pi}\right)^2} \tag{3}$$

$$DMF = \left(\frac{0.05}{\xi}\right)^{0.4} \tag{4}$$

$$\psi_A = \frac{A_{construction}}{A_{service}} \tag{5}$$

$$\psi_D = \frac{D_{construction}}{D_{service}} \tag{6}$$

3. Results

3.1. Periods

Figure 5 shows the variation of the in-service (T_s) and during construction (T_{C1}, T_{C2}, and T_{C3}) periods with the column height for the bridges with span length of 60 m, 130 m, and 200 m. Note that the service condition (S1 to S4 in Fig. 1) is not specified in the plots of Figure 5 because these bridges have the same in-service period due to their distribution of span masses and columns stiffnesses.

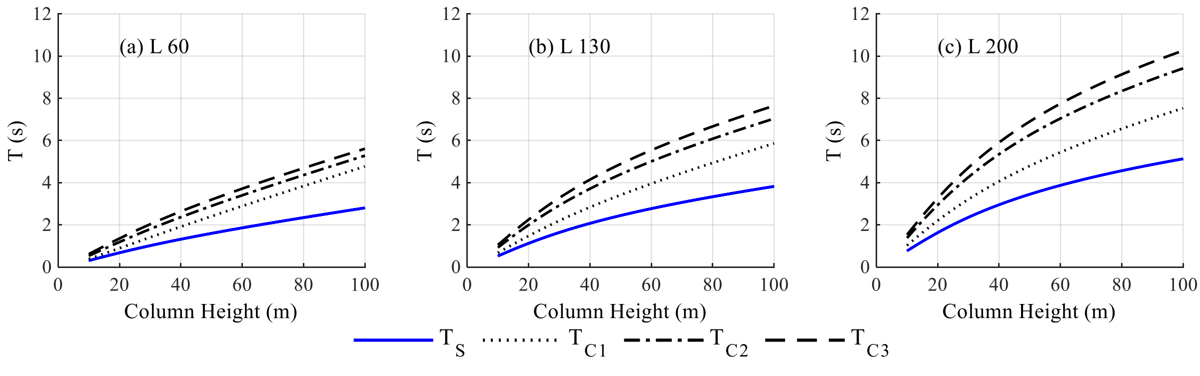

Figure 5. Longitudinal periods of bridges with span length equal to: (a) 60 m, (b) 130 m, and (c) 200 m.

The periods shown in Figure 5 are excessively long when the span length is the longest (200 m) and when the cantilever construction before building the closure segment reaches 100%. These long periods are associated mainly to the high flexibility of the columns that resulted from the calculated geometrical parameters, which are not optimized for design. Although some of the models may not be applicable in practice, the increasing trends observed for the periods allow to study the effects of the parameters mentioned above on the seismic response of the bridges. As seen in Figure 5, the period of all bridges lengthens as the column height increases, which was expected. Additionally, for a given column height, the periods increase as the main span length increases because the girder mass is larger. This is also the reason why the periods in the construction models increase as the cantilever construction progresses.

Figure 5 also shows that the periods of the models during construction are always longer than those of the completed bridges. This is because the mass-to-stiffness ratio of models C1, C2, and C3 is consistently higher than that of models S1 to S4. Among the models, those with span lengths of 60 m showed periods during construction that are close to each other regardless of the column height. This is because the column mass in these models represents a large part of the total mass of the bridge that is used to calculate the period.

3.2. Spectral acceleration demands

The spectral acceleration (ψ_A) and displacement (ψ_D) demand ratios were calculated for each of the three construction stages (C1, C2, and C3) and for the three span lengths (L equal to 60 m, 130 m, and 200 m) studied in this work. The following sections show the results for each of the two damping cases that were investigated.

3.2.1. Damping ratio Case 1

As stated in section 2.2.3 of this paper, a 5% viscous damping ratio was used in Case 1 for the models in service and during construction. The ψ_A ratios for this case are shown in Figure 6. Note that subscripts A-C1 to A-C3 correspond to the spectral ratios for construction stages C1 to C3, respectively. This notation is also used for the displacement spectral ratios discussed subsequently. As seen in the plots, all ψ_A values are smaller than one, meaning that the spectral acceleration demand is always higher for the completed bridges (i.e., in service condition). The comparison of the ψ_A ratios for the models built on soil type A and E allows to conclude that the soil with lower rigidity will result in highest ψ_A. The maximum ψ_A value occurred for the last construction stage (C3) of the bridges with L=60 m. The plots for these models show that ψ_A increases as the column height decreases. This means that the spectral acceleration demand of bridges during construction tends to increase as the stiffness of the bridges increases.

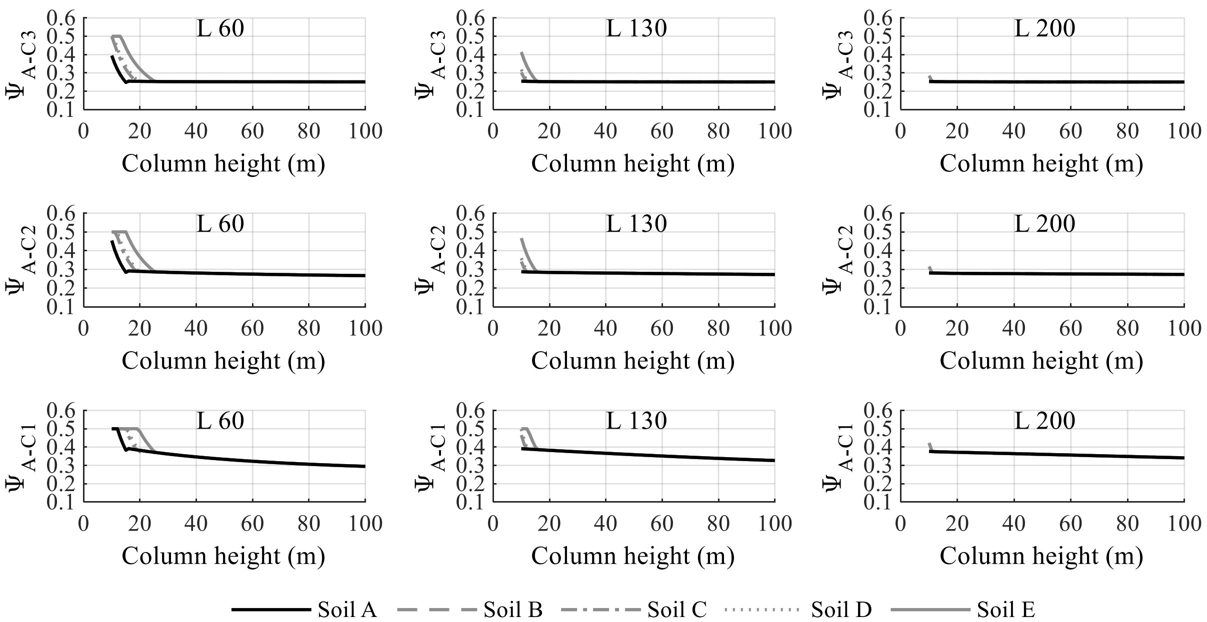

Figure 6. Variation of ψ_A ratios for Case 1.

3.2.2. Damping ratio Case 2

The damping ratios during construction and service for Case 2 were 1.3% and 2%, respectively. Figure 7 shows the calculated ψ_A ratios for this case. As occurred in Case 1, all values of ψ_A are smaller than one, meaning that bridges in-service condition are more vulnerable if the acceleration demand is chosen as the critical seismic response parameter. The ψ_A values in Case 2 are 19% larger than those calculated for the Case 1. This increment is associated to the lower damping ratio of the bridges during construction, which results in slightly larger amplifications of the acceleration demands.

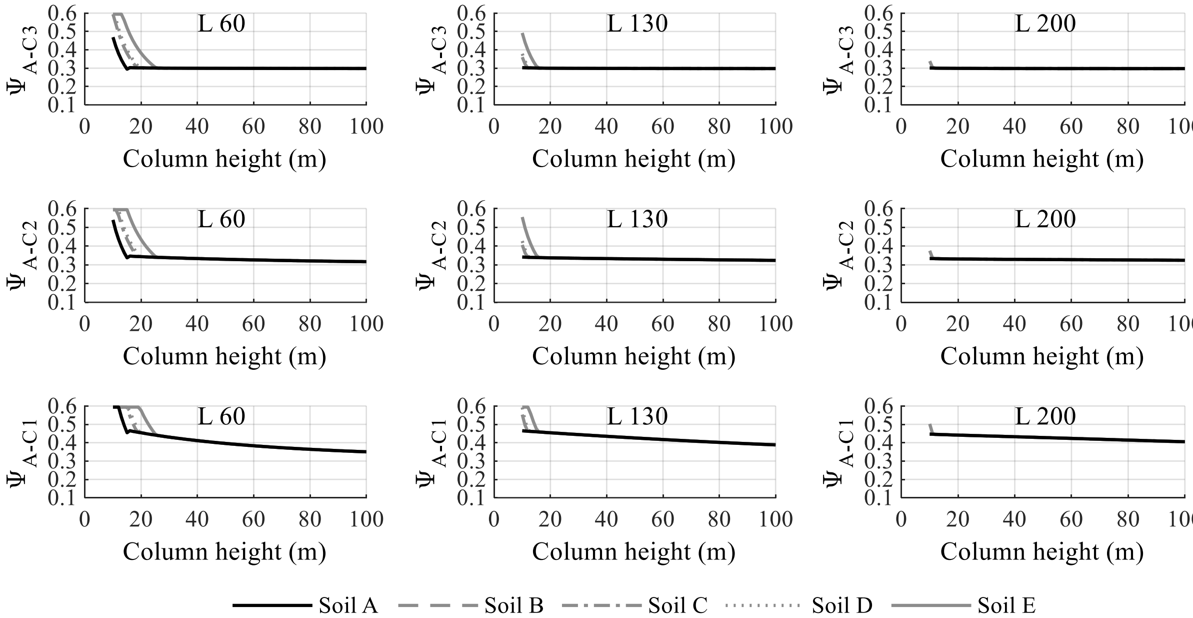

Figure 7. Variation of ψ_A ratios for Case 2.

3.3. Spectral displacement demands

3.3.1. Damping ratio Case 1

The ψ_D ratios for Case 1 are shown in Figure 8. The highest spectral ratios for a fixed span length (L) correspond to ψ_{D-C3}. This means that the construction stage before closing the cantilevers is the most vulnerable condition during construction. This result agrees with the findings by Wilson and Holmes (2007) and Benjumea and Chio Cho (2013) for cable-stayed and extradosed bridges. For construction stage C3, the bridges with span lengths of 60 m, 130 m, and 200 m showed ψ_D values greater than one when the column height is less than 26 m, 16 m, and 11 m, respectively. These bridges have in-service periods of vibration that are less than the short period limit of the response spectrum (*Ts* in Fig. 4), while their periods during construction are longer than that limit. Furthermore, it can be observed in the plots of ψ_{D-C1} that this construction stage is less critical than the in-service conditions, as all ψ_D ratios are smaller than one regardless of the bridge length and column height. Regarding the effect of the soil profile, the ψ_D values increase as the soil stiffness decreases for bridges with in-service periods lesser than the short period limit *Ts*. For the rest of the bridges, the soil type does not significantly influence the displacement demand ratios.

3.3.2. Damping ratio Case 2

Figure 9 shows the displacement ratios (ψ_D) calculated for Case 2, where the damping ratios during construction and service were taken as 1.3% and 2%, respectively. By comparing Figures 8 and 9, it can be concluded that the ψ_D ratios in the latter case are larger than those of Case 1. The average increment in the ψ_D values from Case 1 to Case 2 is 19%, which is equal to the ratio between the DMF for ξ=2% and ξ=1.3%. Due to this amplification, most of the ψ_{D-C2} values are slightly larger than one. This indicates that the seismic displacement demand of the completed bridge, which is associated to the 1033-

years return period earthquake, may be equaled during construction after 70% of the cantilevers have been built even when an earthquake of smaller intensity strikes the structure.

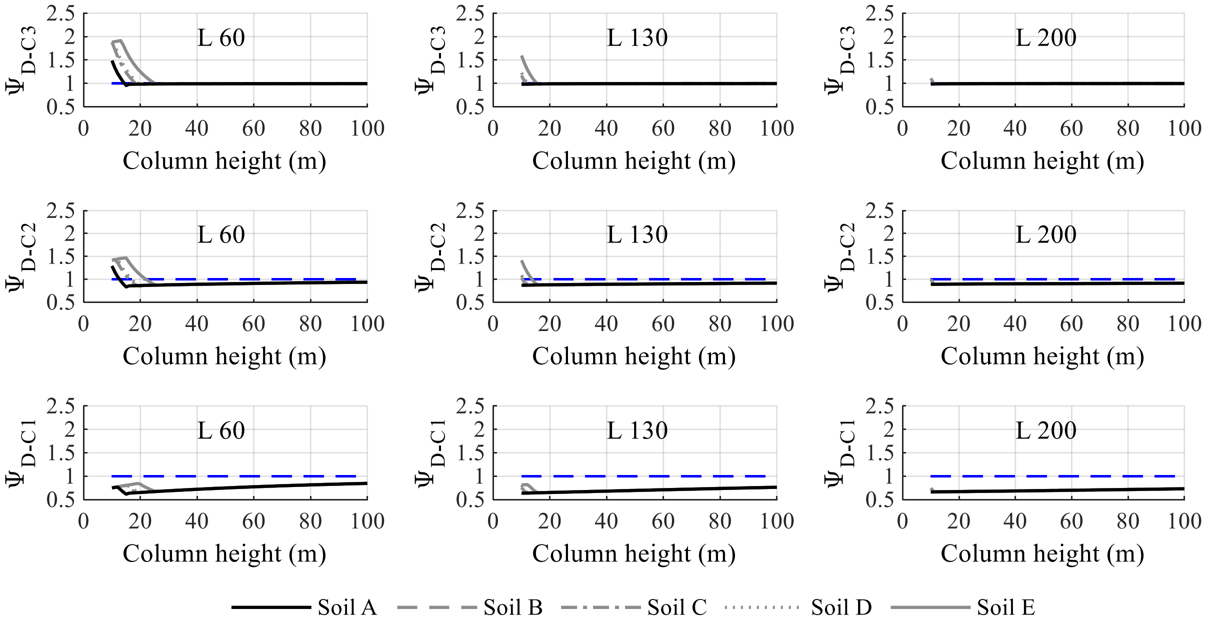

Figure 8. Variation of ψ_D ratios for Case 1.

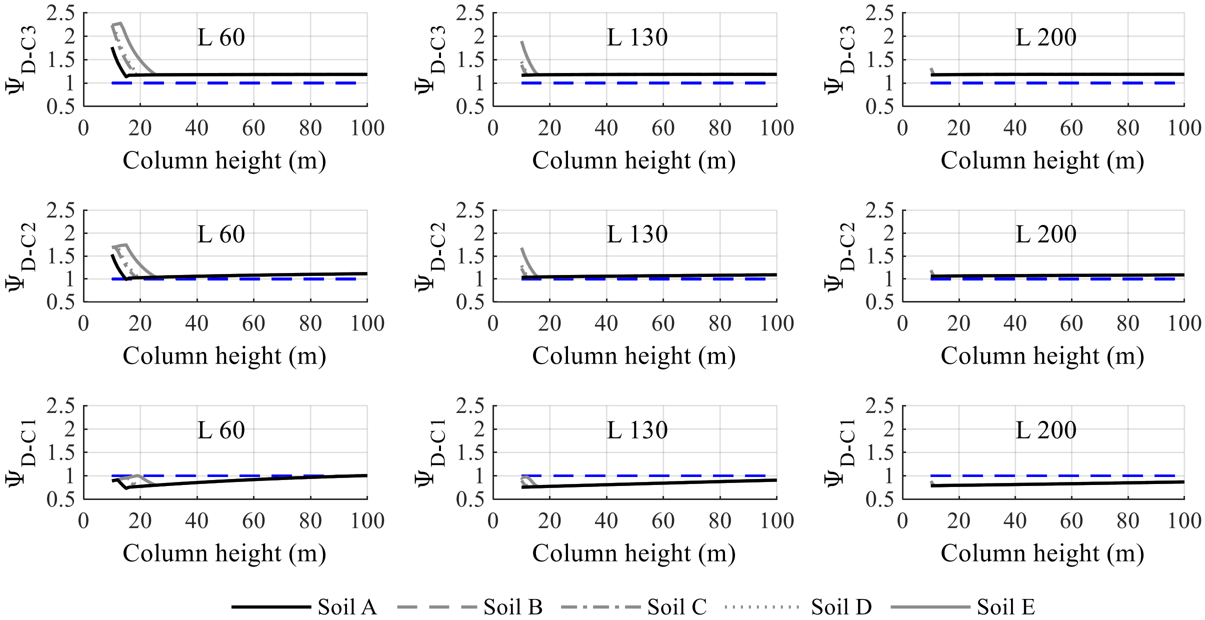

Figure 9. Variation of ψ_D ratios for Case 2.

4. Conclusions

The main conclusions drawn from the results discussed in this paper are:

1. The fundamental period of vibration in the longitudinal direction during construction of balanced cantilever box-girder bridges may be as much as twice the period in service condition, as expected when comparing the cantilever and double-curvature column behavior of the two conditions.
2. As a consequence of the conclusion above, the spectral acceleration demands in balanced cantilever box-girder bridges during construction are smaller than those of the completed structure. Therefore, the

design forces for the construction condition shall be smaller than those for the service condition, based on the traditional force-based approach.
3. Balanced cantilever box-girder bridges like those analyzed in this investigation may be subjected to seismic displacement demands during construction that are equal to or greater than those in service condition. This is an interesting result considering that the amplitude of the pseudo-acceleration response spectra used for the construction analyses were 50% (for damping ratio Case 1) and approximately 41% (for damping ratio Case 2) lower than those of the in-service condition analyses. The most critical cases occurred for bridges with a main span length of 60 m and when the natural period of the completed bridge was shorter than the short period limit of the acceleration design spectrum.
4. When studying the seismic response of bridges during construction, it is recommended to use more representative values of viscous damping ratios obtained by experimental data than the typical 5% suggested by design codes. The more realistic damping ratios of the bridge will avoid underestimating the spectral acceleration and displacement demands on these bridges, especially during construction.
5. As the soil stiffness decreases, the seismic displacement and acceleration demands on the completed and uncompleted bridge increases. The effects due to the soil profile are more noticeable during construction of the bridge.

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References

AASHTO (American association of State Highway and Transportation Officials). (2011). *AASHTO guide specifications for LRFD seismic bridge design* (2nd Ed). Washington, DC.
AASHTO (American association of State Highway and Transportation Officials). (2012). *AASHTO LRFD bridge design specifications* (6th ed.). Washington, DC.
AASHTO (American association of State Highway and Transportation Officials). (2014). *AASHTO LRFD bridge design specifications* (7th ed.). Washington, DC.
AIS (Asociación Colombiana de Ingeniería Sísmica). (2014). *Norma Colombiana de Diseño de Puentes*. Bogotá, Colombia.
Ariñez, F., & Astiz, M. (2012). *Criterios para la optimización del predimensionamiento de puentes de sección cajón*. Madrid: Universidad Politécnica de Madrid.
Bardakis, V., & Fardis, M. (2011). Nonlinear dynamic v elastic analysis for seismic deformation demands in concrete bridges having deck integral with the piers. *Bulletin of Earthquake Engineering*, *9*(2), 519–535. https://doi.org/10.1007/s10518-010-9203-9
Bavraktar, A., Nur Kudu, F., Sumerkan, S., Betül, D., & Akköse, M. (2020). Near-fault vertical ground motion effects on the response of balanced cantiliver bridges. *Bridge Engineering*, *173*(1), 17–33. https://doi.org/https://doi.org/10.1680/jbren.19.00007
Benjumea, J., & Chio Cho, G. (2013). Seismic vulnerability assessment of extradosed bridges during cantilever construction. *Revista Ingeniería de Construcción*, *28*(2), 125–139. https://doi.org/10.4067/s0718-50732013000200002
Chang, K., Mo, Y., Chen, C., Lai, L., & Chou, C. (2004). Lessons Learned from the Damaged Chi-Lu Cable-Stayed Bridge. *Journal of Bridge Engineering*, *9*(4), 343–352. https://doi.org/10.1061/(asce)1084-0702(2004)9:4(343)
Hernandez, W., Viviescas, A., & Riveros-Jerez, C. A. (2020). Verifying of the finite element model of the bridge based on the vibration monitoring at differente stages of construction. *Archives of Civil Engineering*, *66*(1), 25–40. https://doi.org/10.24425/ace.2020.131772
Li, X., Li, Z. X., & Crewe, A. (2018). Nonlinear seismic analysis of a high-pier, long-span, continuous RC frame bridge under spatially variable ground motions. *Soil Dynamics and Earthquake Engineering*, *114*(May), 298–312. https://doi.org/10.1016/j.soildyn.2018.07.032
Mantterola, J. (2006). *Puentes: apuntes para su diseño, calculo y construcción*. Madrid: Colegio de ingenieros de caminos, canales y puertos.
Ntotsios, E., Karakostas, C., Lekidis, V., Panetsos, P., Nikolaou, I., & Papadimitriou, C. (2009). *Structural identification of Egnatia Odos bridges based on ambient and earthquake induced vibrations*. 485–501. https://doi.org/10.1007/s10518-008-9074-5
Panetsos, P., Ntotsios, E., & Papadimitriou, C. (2008). Structural health monitoring of a ravine bridge of Egnatia Motorway during construction. *Proceedings of the 4th European Workshop on Structural Health Monitoring*, (June), 259–266.
Wilson, J., & Holmes, K. (2007). Seismic vulnerability and mitigation during construction of cable-stayed bridges. *Journal of Bridge Engineering*, *12*(3), 364–372. https://doi.org/10.1061/(ASCE)1084-0702(2007)12:3(364)
Xinle, L., Huijuan, D., & Xi, Z. (2007). *Engineering characteristics of near-fault vertical ground motions and their effect on the seismic response of bridges*. *6*(4), 1–6.