

Chapter 5

Parametric Study

5.1. Objective

The objective of this parametric study is to examine common substructure configurations to determine the influence of the new LRFD Guidelines on column longitudinal and transverse (confinement) reinforcement. US Customary units were used for all the dimensions and calculations in this parametric study because US Customary units are used by the Virginia Department of Transportation and more commonly used than Metric units in this country.

5.2. Bridge Structure

The RISA 3D model of the bridge is shown in Figure 5.1. In this model, unlike

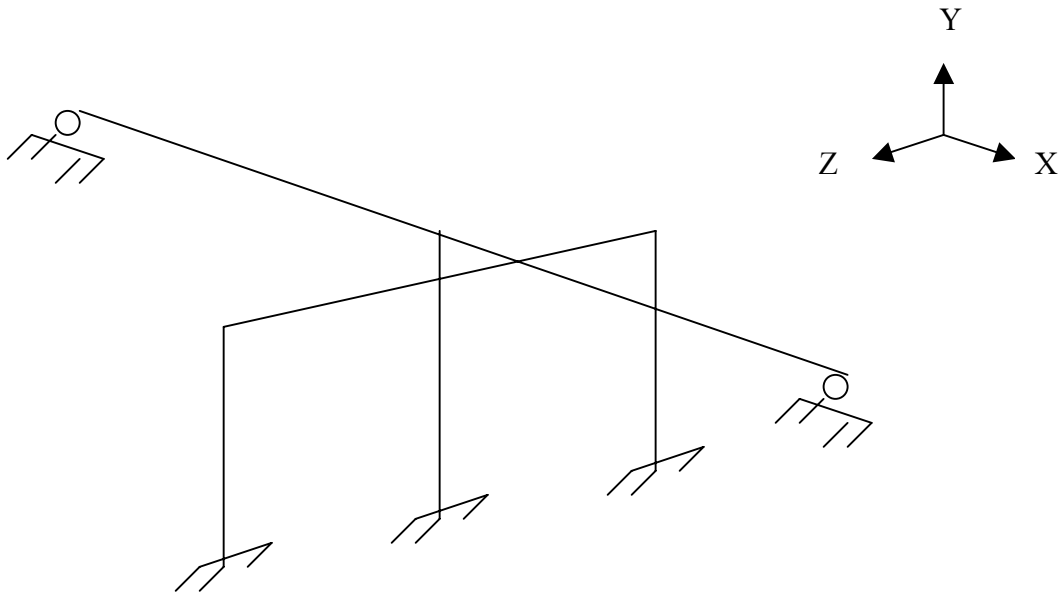


Figure 5.1. The RISA 3D model of the bridge used in this parametric study.

the RISA 3D models for the bridges in Chapters 3 and 4, the longitudinal (x-direction) supports at the abutments were released. Thus the bridge had sliding bearings that would allow only longitudinal movement (x-direction) at the abutments.

The section properties of the superstructure used in this parametric study was taken from the average section properties of the two steel girder bridges (West Bound and East Bound) analyzed in Chapter 4. Therefore the section properties for the superstructure in this parametric study are:

$$Area = (\text{West Bound Area} + \text{East Bound Area}) / 2$$

$$Area = \frac{899in^2 + 1114in^2}{2}$$

$$Area \approx 1007in^2$$

$$I_{xx} = (\text{West Bound } I_{xx} + \text{East Bound } I_{xx}) / 2$$

$$I_{xx} = \frac{373,000in^2 + 521,000in^2}{2}$$

$$I_{xx} = 447,000in^2$$

$$I_{yy} = (\text{West Bound } I_{yy} + \text{East Bound } I_{yy}) / 2$$

$$I_{yy} = \frac{25,600,000in^2 + 50,500,000in^2}{2}$$

$$I_{yy} = 38,050,000in^2$$

The substructure used in this parametric study is shown in Figure 5.2. The bent has three 36-in.-diameter columns at 20 ft center-to-center. The pier cap beam is 45 in. wide and 50 in. deep. The height of the columns was one of the parameters in this study, and the five different column heights were 20 ft, 25 ft, 30 ft, 35 ft and 40 ft. For each column, f_c' was 3600 psi (25 MPa). The reinforcement ratio for each column was 1.5%. The bridge had two spans, and span length was also a parameter in this study. The four different span lengths were 80 ft, 90 ft, 100 ft and 110 ft.

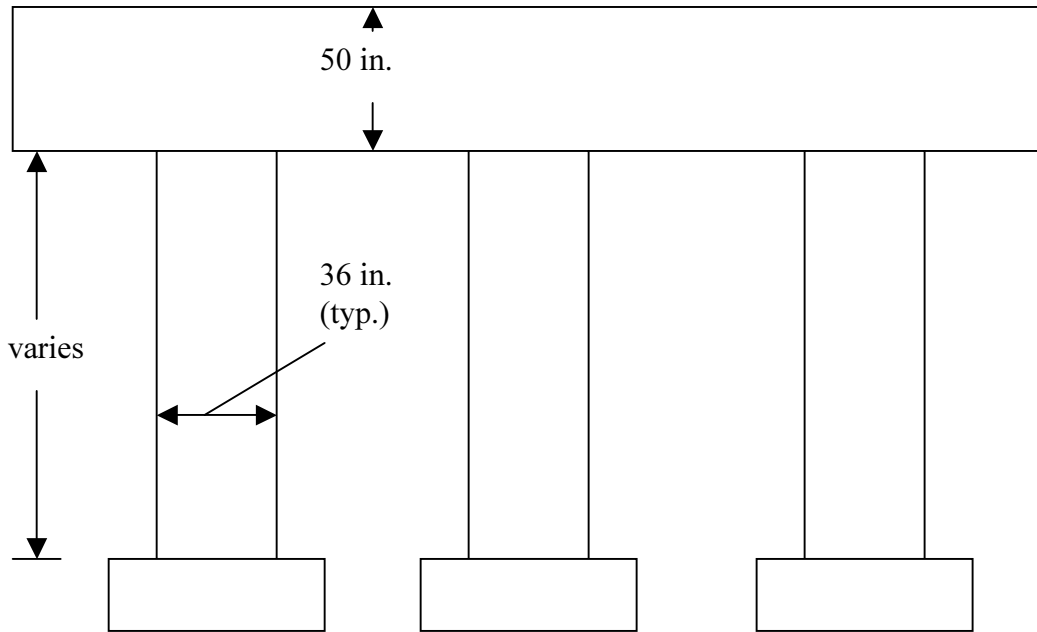


Figure 5.2. The substructure of the bridge used in this parametric study. Column-to-column spacing = 20 ft. This figure was not drawn to scale.

5.3. Bridge Stiffness

All the calculations for this parametric study are provided in Appendix XXIII. To determine the periods of vibration of the two bridges, the stiffness values of the bridges in the transverse and longitudinal directions were determined. The stiffness in the transverse direction was obtained by applying the force P to the substructure as shown in Figure 5.3. Cracked section properties were used for the columns. To calculate the cracked section properties, the superstructure was assumed to be 12 k/ft, and the weight of the pier cap beam was assumed to be 120 kips. The ratio $P/(f_c' A_g)$ was calculated for each combination of span length and column height. Then the ratio I_e/I_g , which was obtained from using the ratio $P/(f_c' A_g)$ and Figure 3.12, was also calculated for each combination of span length and column height. The four different I_e/I_g ratios for every column height were averaged and the I_e average was used for all the columns of the five bridge models with that column height to determine the stiffness in the transverse direction. The stiffness in the transverse direction was calculated by using equation 5-1:

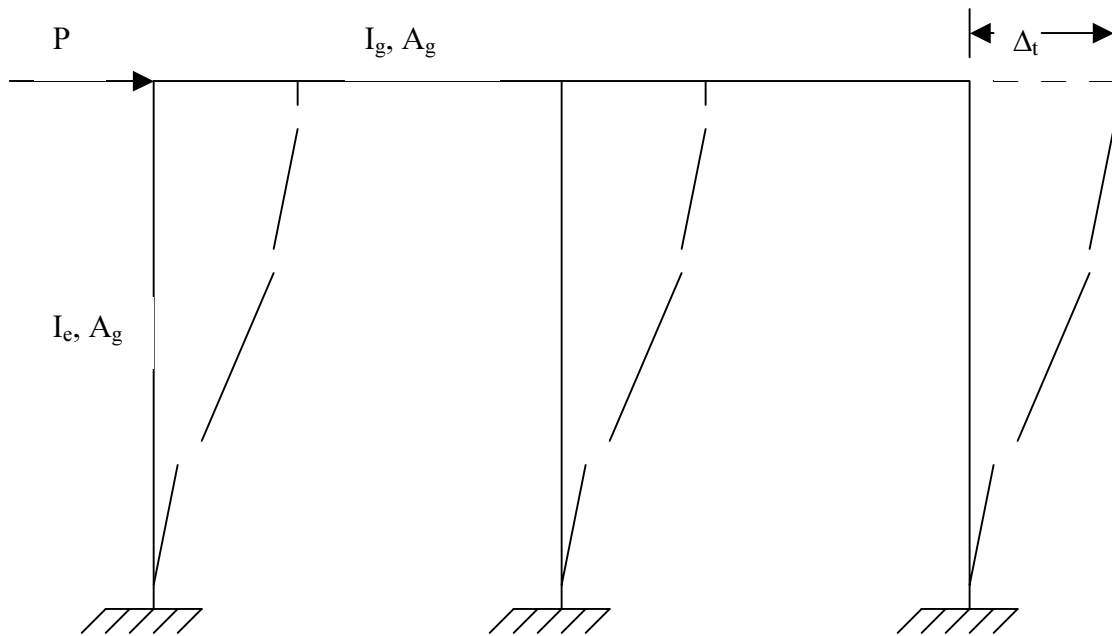


Figure 5.3. The loading to the substructure to get the stiffness in the transverse direction.

$$K_{tb} = \frac{P}{\Delta_t} \quad (5-1)$$

K_{tb} = the stiffness in the transverse direction

P = the magnitude of the force in the transverse direction

Δ_t = the deflection in the transverse direction

The stiffness in the longitudinal direction was obtained by applying the force P to a single cantilever column, whose section properties represent that of three columns of the substructure. This is shown in Figure 5.4. The stiffness in the longitudinal direction was calculated by using equation 5-2:

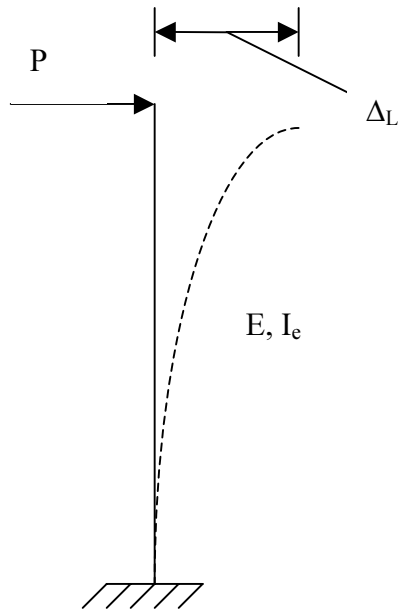


Figure 5.4. The loading to the substructure, modeled as a cantilever column, to get the stiffness in the longitudinal direction.

$$K_L = \frac{P}{\Delta_L} = \frac{3EI_e}{L^3} \quad (5-2)$$

K_L = the stiffness in the longitudinal direction

P = the magnitude of the force in the longitudinal direction

Δ_L = the deflection in the longitudinal direction

E = the modulus of elasticity of concrete

I_e = the total cracked moment of inertia of the three columns of the substructure

L = the length (height) of the cantilever column

= height of the column + (0.5 × pier cap beam height)

Equation 5-2 comes from the deflection formula for a cantilever column, which is

$$\Delta_L = \frac{PL^3}{3EI}$$

5.4. Periods of Vibration

The period of vibration in the transverse and longitudinal were determined for each combination of span length and column height.

The period of vibration in the transverse direction was determined by using the configuration shown in Figure 5.5. A 1 k/ft uniformly distributed load was applied to a

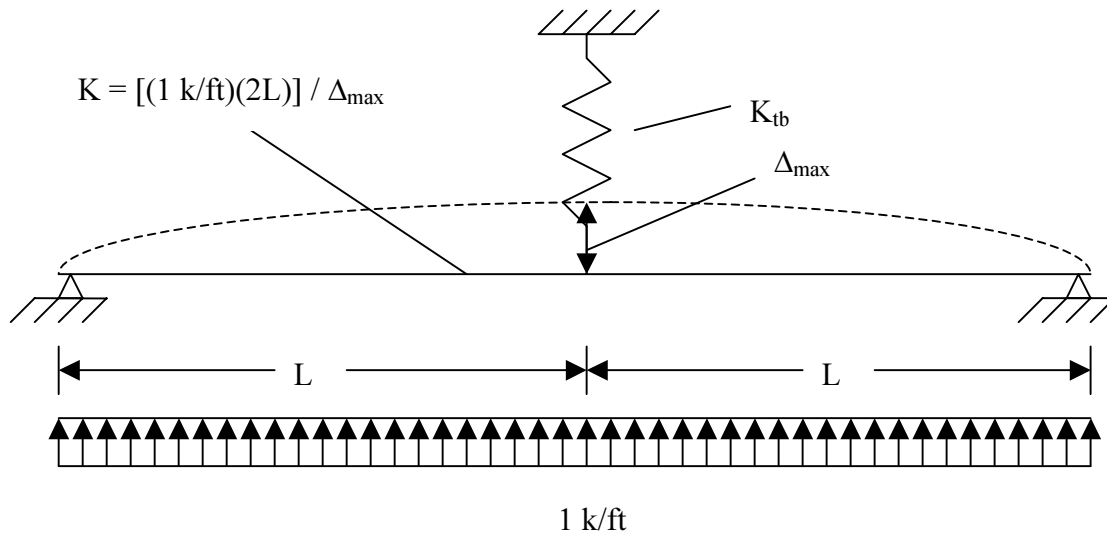


Figure 5.5. The loading to obtain the period of vibration of the bridge in the transverse direction.

simply-supported two-span bridge with a spring, which has stiffness of K_{tb} defined in section 5.3, attached to the midpoint of the bridge. The maximum deflection, Δ_{max} , was then used to calculate the period of vibration in the transverse direction using equation 5-3:

$$T_t = 2\pi \sqrt{\frac{W}{gK_t}} \quad (5-3)$$

T_t = period of vibration in the transverse direction

W = weight of the superstructure and pier cap beam (columns excluded)

g = gravitational acceleration = 386 in/sec²

$$K_t = \frac{(1k / ft)(2l)}{\Delta_{\max}}$$

l = length of one span of the bridge

The period of vibration in the longitudinal direction was calculated using equation 5-4 for each combination of span length and column height.

$$T_L = 2\pi \sqrt{\frac{W}{gK_L}} \quad (5-4)$$

W = weight of would have been larger the superstructure and pier cap beam (columns excluded)

g = gravitational acceleration = 386 in/sec²

K_L = bridge stiffness in the longitudinal direction

The periods of vibration in the transverse and longitudinal directions for each combination of span length and column height are given in Tables 5.1 and 5.2, respectively.

It is important to note that the height of the columns made no difference for the period of vibration in the transverse direction. This was the case because in this parametric study, the bridge was modeled with the superstructure much stiffer than the columns. If the columns had been modeled much stiffer, Δ_t in Figure 5.3 would have been much smaller, K_{tb} would have been much larger, Δ_{\max} in Figure 5.5 would have been smaller, K_t in equation 5-3 would have been larger and T_t (the period of vibration in the transverse direction) would have been different for every column height (the higher the column, the longer the period of vibration).

It is also important to note that the periods of vibration in the longitudinal direction are very high. This was the case because K_L , the bridge stiffness in the longitudinal direction, is very low. This makes sense, since the bridge is stiffer in the transverse direction than in the longitudinal direction. Another important thing to note is

that the periods of vibration in the longitudinal direction did not increase proportionally with (column height)^{3/2} as may have been implied by Figure 5.4 and Equation 5-4, because as stated in the definitions of terms in equation 5-2, the L in equation 5-2 is the column height plus half the pier cap beam height, instead of just the column height.

5.5. Equivalent Earthquake Loads

To determine the equivalent earthquake loads, the design response spectrum curve was drawn. This parametric study investigates the effects of earthquake loads in three

Table 5.1. The periods of vibration in the transverse direction (sec.)

		Column Height				
		20 ft	25 ft	30 ft	35 ft	40 ft
Span	80 ft	0.130	0.130	0.130	0.130	0.130
	90 ft	0.165	0.165	0.165	0.165	0.165
Length	100 ft	0.200	0.200	0.200	0.200	0.200
	110 ft	0.240	0.240	0.240	0.240	0.240

Table 5.2. The periods of vibration in the longitudinal direction (sec.)

		Column Height				
		20 ft	25 ft	30 ft	35 ft	40 ft
Span	80 ft	1.237	1.680	2.166	2.692	3.254
	90 ft	1.308	1.776	2.290	2.846	3.440
Length	100 ft	1.375	1.867	2.407	2.992	3.617
	110 ft	1.439	1.954	2.519	3.131	3.785

locations: Vienna, VA, where the seismic risk is low, Richmond, VA, where the seismic risk is moderate, and Bristol, VA, where the seismic risk is high. The design response spectrum curves at Vienna, Richmond and Bristol for soil class B are shown in Figures 5.6, 5.7 and 5.8, respectively.

The equivalent earthquake loads in the transverse direction were calculated for every span length (80 ft, 90 ft, 100 ft and 110 ft) at every location (Vienna, Richmond and Bristol) by using equation 5-5:

$$p_e = \frac{S_a W}{L} \tag{5-5}$$

p_e = equivalent earthquake load in the transverse direction

S_a = spectral acceleration from the design response spectrum curve

W = weight of the superstructure and the pier cap beam

L = the length of the bridge

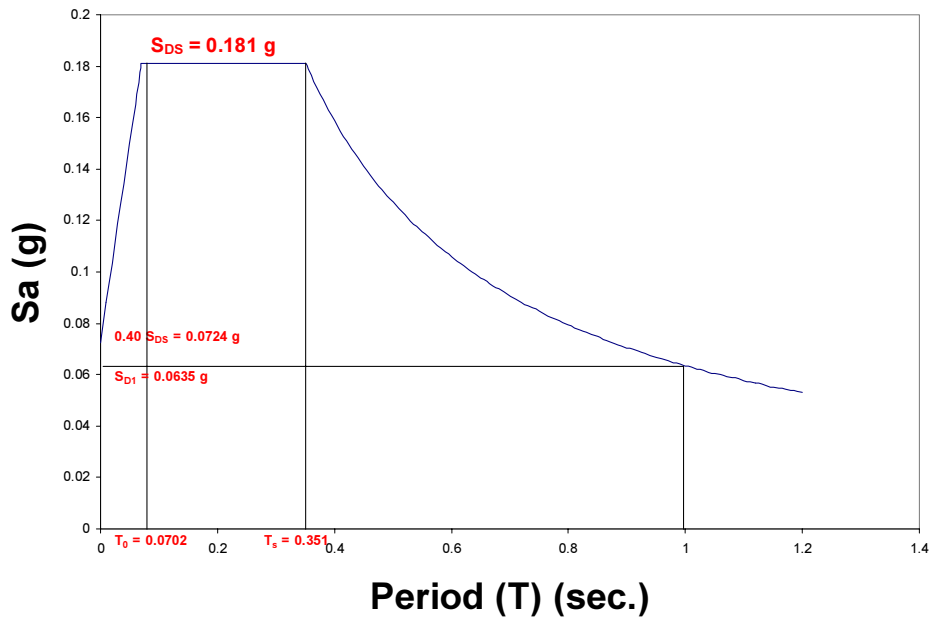


Figure 5.6. The Design Response Spectrum Curve for Vienna, VA.

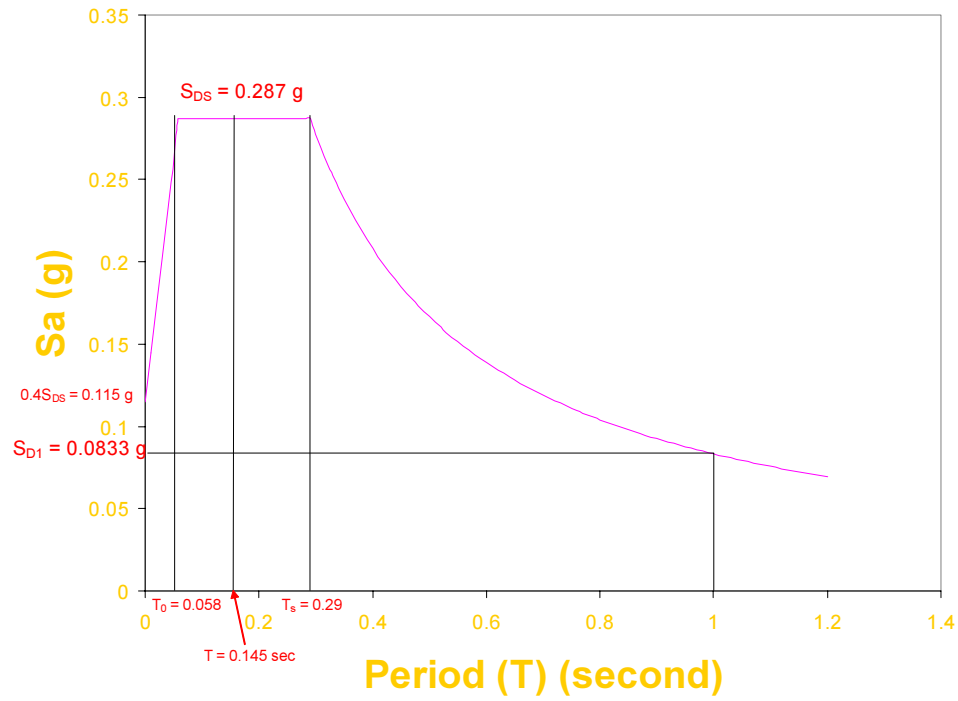


Figure 5.7. The Design Response Spectrum Curve for Richmond, VA.

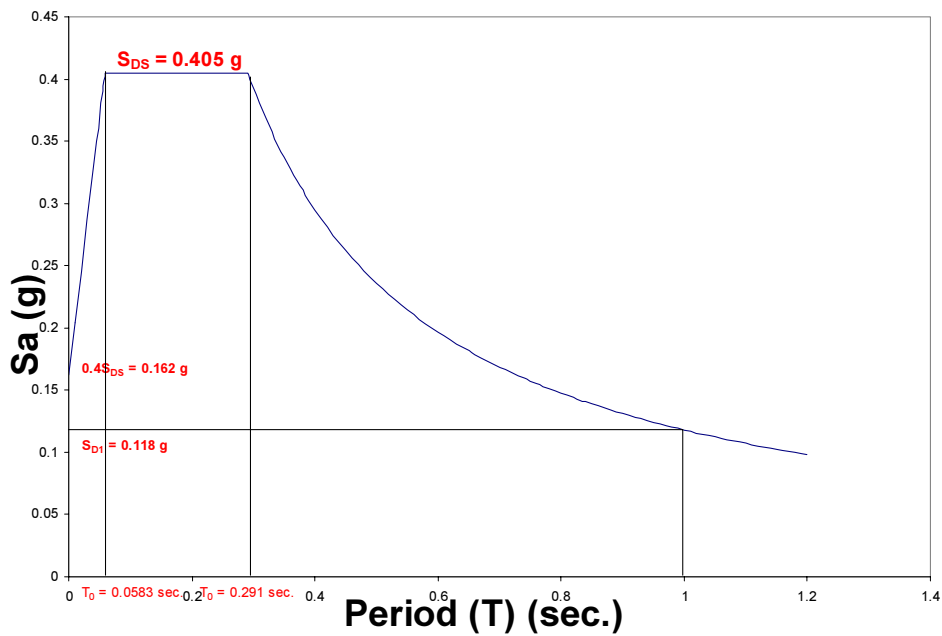


Figure 5.8. The Design Response Spectrum Curve for Bristol, VA.

The equivalent earthquake loads in the longitudinal direction were calculated for each combination of span length and column height at every location by using equation 5-6:

$$P = S_a W \quad (5-6)$$

P = equivalent earthquake load in the longitudinal direction

S_a = spectral acceleration from the design response spectrum curve

W = weight of the superstructure and the pier cap beam

5.6. Column Interaction Diagram

The column moment and axial load were calculated for each combination of span length and column height at all three locations (Vienna, Richmond and Bristol) by using equation 5-7:

$$P = 1.0DL + 0.5LL + 1.0 \frac{EQ}{R} \quad (5-7)$$

P = the total combined effects from dead, live and earthquake loads

DL = the dead load effects

LL = the live load effects

EQ = the earthquake load effects

R = the base response modification factor from Table 3.9 = 1.5

The column moments and axial loads were plotted and compared to the column interaction diagram, which was plotted using straight lines to connect the important points of the curve. The column interaction diagrams were plotted using straight lines instead of the actual curved line to ensure that no points would fall just inside the curve interaction diagram, which would mean that some columns were only barely satisfactory. The comparisons for Vienna, Richmond and Bristol are shown in Figures 5.9, 5.10 and

5.11, respectively. As the three figures show, all the Vienna and Richmond bridges are adequate to sustain the design dead, live and earthquake loads, while some of the Bristol bridges are not. The inadequate Bristol bridges have short column heights and long spans, which makes sense, since they have shorter periods of vibration, which result in larger values of S_a , and therefore create larger equivalent earthquake forces. Furthermore, those bridges have larger total weight. In order to make them adequate, the reinforcement ratio has to be increased from 1.5% to 2%, as shown in Figure 5.11.

In conclusion, the periods of vibration in the transverse direction were not affected by the height of the columns, but they increased as the bridge spans became longer. As explained in section 5.4, that was the case because the bridge was modeled with the superstructure much stiffer than the columns, which was the opposite of the case with the bridges analyzed in Chapters 3 and 4, where the columns were much stiffer than the superstructure. The periods of vibration in the longitudinal direction increased as the columns became taller, and they also increased as the bridge spans became longer. All the columns, except the Bristol bridges with short columns and long spans, had enough longitudinal reinforcement. The longitudinal reinforcement for the Bristol bridges with short columns and long spans has to be increased from 1.5% to 2%. It is important to note that in this parametric study, unlike the bridges analyzed in Chapters 3 and 4, the column capacity was not much higher than the maximum axial loads and moments in the columns, as shown in Figures 5.9, 5.10 and 5.11. This was due to the release of longitudinal constraints (x-direction) at the ends of the superstructure (at the abutments) with the use of sliding bearings, which made the columns sustain more loads from the earthquake forces in the longitudinal direction.

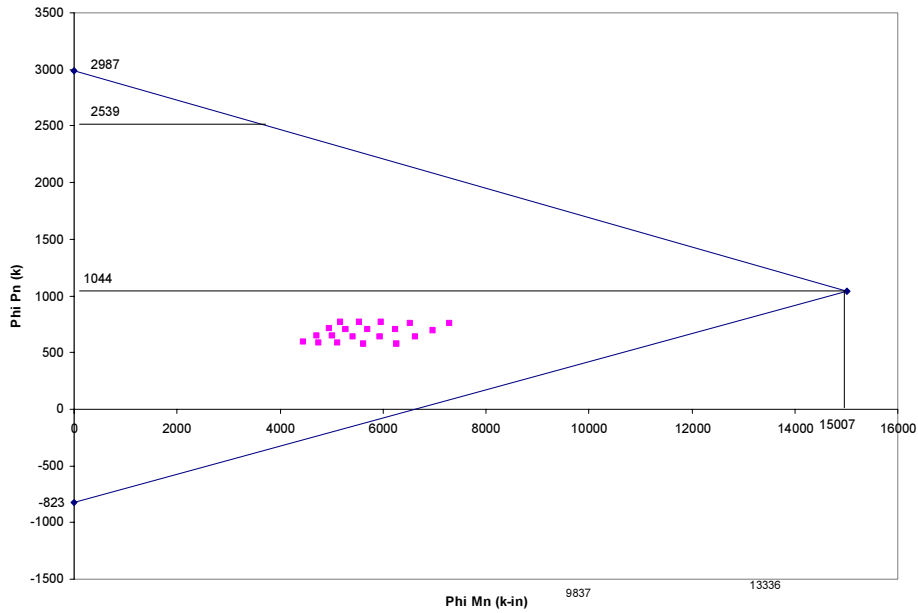


Figure 5.9. The comparison between the column interaction diagram and the axial loads and moments of the column for Vienna, VA. The grey points are the factored axial loads and moments in the columns.

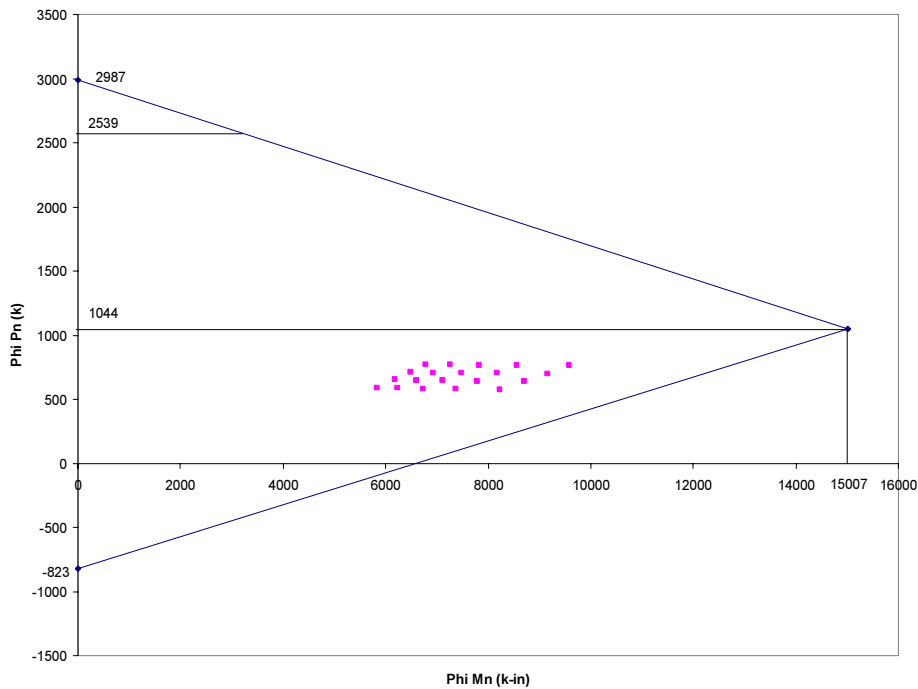


Figure 5.10. The comparison between the column interaction diagram and the axial loads and moments of the column for Richmond, VA. The grey points are the factored axial loads and moments in the columns.

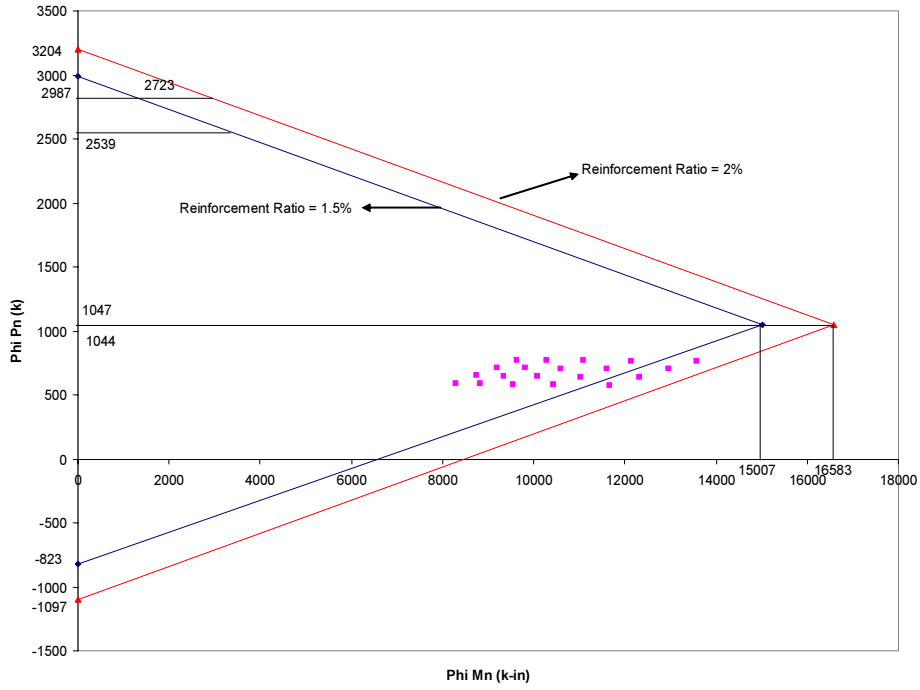


Figure 5.11. The comparison between the column interaction diagram and the axial loads and moments of the column for Bristol, VA. The grey points are the factored axial loads and moments in the columns.