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DESIGN AND CONSTRUCTION OF A CONCRETE GIRDER BRIDGE WITH SEISMIC PINS AT THE TOP OF COLUMNS

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ABSTRACT

Seismic hinges or pins are becoming more prevalent in the design of concrete bridge columns. At one end of the column the full moment connection is replaced by a very short small-diameter concrete column or a concrete-filled steel tube. This short column section has relatively little moment resistance and thus acts as a hinge under seismic loads. Seismic pins lengthen the natural period of the structure, thus reducing seismic demands, and essentially eliminate seismic moments at one end of the columns.

For the design of a 12-span girder bridge located in Langley, British Columbia, seismic pins made of 406 mm diameter concrete-filled steel tubes were placed at the top of columns. The bridge is divided into three 4-span continuous segments with column heights ranging from 4.7 m to 8.9 m. Placing the pins at the top of columns has a number of advantages over placing them at the base, or using fixed-ended columns. Under earthquake loads, the columns act as cantilevers. This reduces the impact from the numerous variations in span lengths, column spacing and bent orientation, and makes the response similar in both transverse and longitudinal directions despite a significant horizontal curve in the middle of the bridge.

In addition to improving the seismic response of the bridge, using the pins allowed the superstructure to be designed separately from the substructure. This was crucial for the timely completion of this design-build project.

Introduction

The use of pins in columns is becoming more prevalent as an alternative to forming a plastic hinge under seismic demands. The pins are small-diameter column stubs with relatively low moment capacity but large flexural ductility. They enable a simpler less congested connection between the column and the foundation or between the column and the cap beam. Caltrans Memo to Designers 6-1 (October 2001) advocates the use of column pins at the base of columns to limit the transfer of seismic demands to the foundations. For the 204th St. Viaduct project discussed in this paper, and for the Sound Transit Link Light Rail Project in Seattle, column pins were placed at the top of columns. Fig. 1 shows two different types of column pins: a reinforced concrete pin and a Concrete Filled Tube (CFT).

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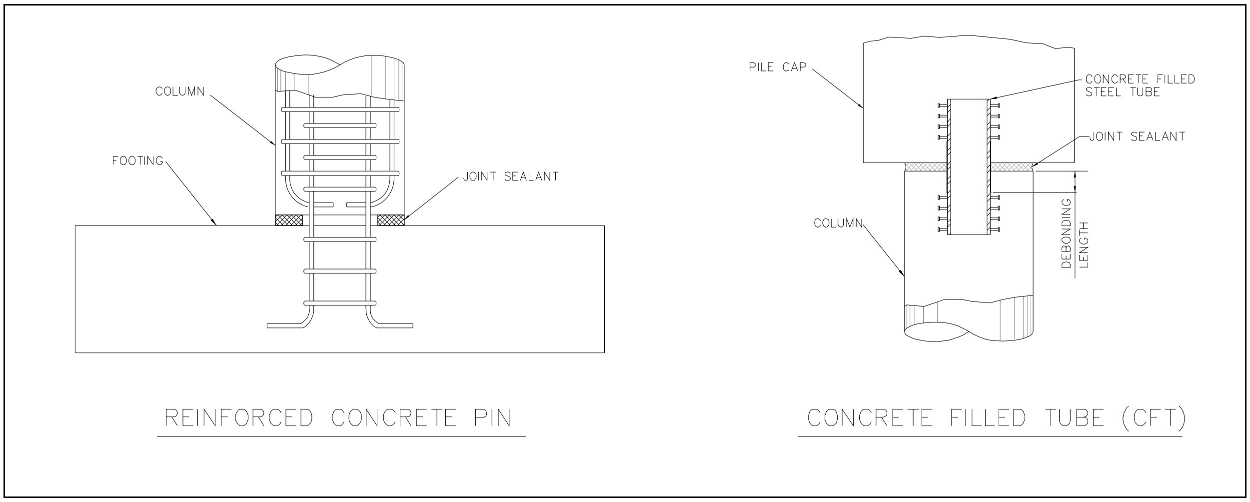


Figure 1. Two types of column pins.

Compared to a rigid frame, pinned columns make the structure more flexible, which lengthens the natural period and reduces earthquake demands. The trade-off is larger displacements and increased moment at the fixed end of the pinned column. Pins at the bottom of the column can significantly reduce the plastic overstrength moment transferred to the foundation, allowing these to be smaller and less expensive. Alternatively, pins at the top reduce moments transferred to the cap beam and eliminate the congestion typically associated with these moment connections. Pins at the top also reduce differences between transverse and longitudinal responses, as the columns act essentially as cantilevers.

With pins between the columns and the superstructure, the superstructure can be designed essentially independently of the substructure. Design and construction of the substructure can begin as soon as a reasonable estimate of the superstructure weight is available, and can be completed before the superstructure design is complete. This was an important benefit for the 204th St. project, which had a very tight design-build schedule.

The advantages and disadvantages of using column pins, and whether they should be at the top or bottom, depends on a number of factors such as flexibility of the soil, characteristics of the design ground motions, stiffness of the columns and pier caps, and P-delta effects. The 204th St. Viaduct provides an example of a actual bridge with columns pinned at the top, where both design and construction considerations are taken into account.

204th Street Viaduct

The 204th Street viaduct is a 12-span, 420 m long, concrete bridge located in Langley, British-Columbia. This 4-lane urban arterial route passes over a highway, a set of 3 rail tracks, a spur line, a retention pond/marsh, and straddles an industrial access road for 4 spans. The bridge is a precast concrete girder bridge on post-tensioned cap beams. Each bent has two 1.2 m diameter columns, sitting on a square pilecap with 4 – 610 mm diameter steel pipe piles. The piles are founded in deep, relatively soft clay. The deck is formed of precast panels with a cast-in-place topping. Fig. 2 shows a plan of the whole bridge and a typical bent elevation.

The bridge is divided into three units of 4 spans, with expansion joints between units. At interior piers, the superstructure is made continuous by casting the girders into the cap beams. As a result of the congested site, only two of the thirteen bents are the same. To accommodate the access road underneath, the column spacing in unit 1 varies from 13 m to 18 m. Unit 2 crosses the rail tracks: the bents adjacent to the tracks are parallel to the tracks and skewed 50 degrees to the bridge axis. Unit 2 also has a reversing horizontal curve (“S” curve), which adds to the complexity of the structure. Unit 3 is mostly straight and has a constant column spacing of 13 m, but different span lengths including a 38 m end span to clear the highway below. Overall, span lengths vary from 31 m to 39 m, with one skewed span where girders range from 36 m to 23 m within the span. Column heights vary from 4.7 m to 8.9 m to accommodate grade changes of up to 9% and cross-falls reaching 2.7%.

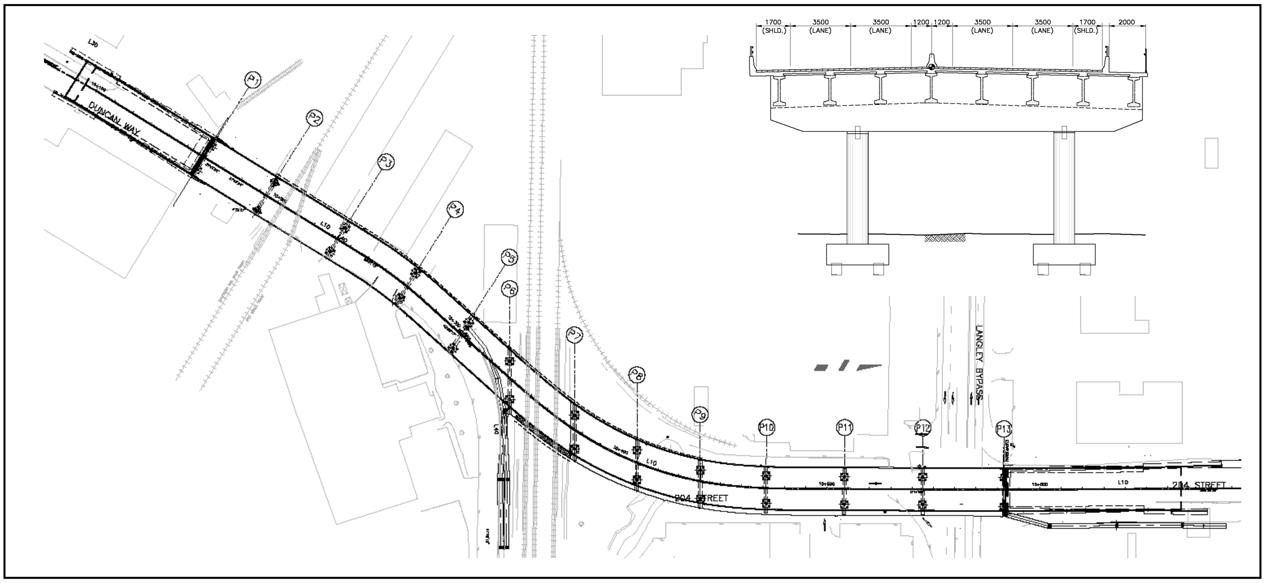


Figure 2. Overall plan and typical bent cross-section for the 204th Street Viaduct.

With pins at the top, the columns act essentially as cantilevers under earthquake loads and thus have the same stiffness properties in all directions. In theory, this eliminates the influence of the horizontal curve, the different bent orientations, and the different column spacings. It also makes the response of all three units the same in both transverse and longitudinal directions, and reduces any torsion effects. In practice, the different column heights and lateral – but not longitudinal – restraint at the expansion joints leads to some non-uniformity in the response. Nonetheless, effects of the complicated geometry are minimized.

The column pins are made with thick wall steel tubes (Fig. 3). The pins are all 406 mm diameter, but the wall thickness is varied depending on the axial load at each bent. Table 1 summarizes the three different steel tubes used on the 204th Street viaduct. Shear studs are welded to the outside of the pins to help transfer loads into the column below and the pier cap above. The pipes are filled with the same 35 MPa concrete as the columns, at the same time as the columns are cast.

Figure 3. Photo of a steel tube used in the 204th Street Viaduct project.

Table 1 Summary of steel tubes used for the 204th Street Viaduct project.

Designation	16" sched. 120	16" sched. 100	16" sched. 80
f_y	345 MPa	345 MPa	345 MPa
D	406 mm	406 mm	406 mm
t	30.9 mm	26.2 mm	21.4 mm
D/t	13.0	15.5	19.0
Total length	1475 mm	1275 mm	1275 mm

Design and Analysis

Seismic design of the 204th St. Viaduct follows the requirements for "Other" structures under Section 4 of the Canadian Highway Bridge Design Code, S6-00 (CSA 2000). Due to the presence of deep relatively soft clays, a site-specific design spectra generated from the Fourth Generation Seismic Hazard Maps of Canada (Geological Survey of Canada 2004) was used instead of the code spectra. The resulting design spectra (Fig.4) shows increased accelerations at short periods and a plateau in the 1.0 to 1.6 second range caused by the characteristics of the site.

With columns pinned at the top, preliminary design can be done very easily by hand or with a spreadsheet. The stiffness at each column is determined by assuming a cantilever column in combination with foundation springs for translation and rotation at the pilecap level. Foundation spring stiffnesses are based on the displacement corresponding to the plastic overstrength moment of the column. Transversely, the bent stiffness is the sum of the two columns, and is associated with its tributary mass. Longitudinally, the stiffness of each bent in a unit is summed, and is associated with the total unit mass. The natural period in each direction is then simply:

$$T = 2\pi \cdot \sqrt{\frac{M}{K}} \qquad (1)$$

where M is the bent or unit mass and K is the bent or unit total stiffness.

The corresponding acceleration is obtained from the design spectra (assuming Single Degree of Freedom response), and the resulting horizontal elastic force is the mass times the acceleration. The simplicity of this approximate method makes it very easy to iterate the initial design and determine the correct column diameter and reinforcement ratio. The simplified analysis indicated average longitudinal and transverse periods of 2.20 and 2.40 seconds. The resulting preliminary design had 1.2 m diameter columns with reinforcement ratios ranging from 0.8 to 2.5% and a maximum plastic overstrength moment of 10MN. The maximum elastic displacement was estimated as 350 mm. Because of the relatively large displacements, P-delta effects were included as an additional static moment, which is effectively a reduction in the force reduction factor (R factor).

For the final design, a 3-dimensional linear elastic model was created in SAP2000 (Computers and Structures, 1999). All three units were modeled together so that connectivity at expansion joints could be evaluated. The model includes foundation springs, columns with cracked section stiffness, and short elements representing the column pins. The superstructure is modeled with spine elements. Modal analysis indicated average longitudinal and transverse periods of 2.30 and 2.21 seconds respectively. A multi-mode response spectra analysis was used to determine elastic demands and displacements. The analysis indicated only the first horizontal and translational modes (for each unit) contribute significantly to the demands, confirming behaviour close to a Single Degree of Freedom system. Maximum elastic displacement from the model was 340 mm. Final design based on the response spectra analysis consists of 1.2 m diameter columns with reinforcement ratios ranging from 0.9 to 2.4% and a maximum plastic overstrength moment of 9.1 MN.

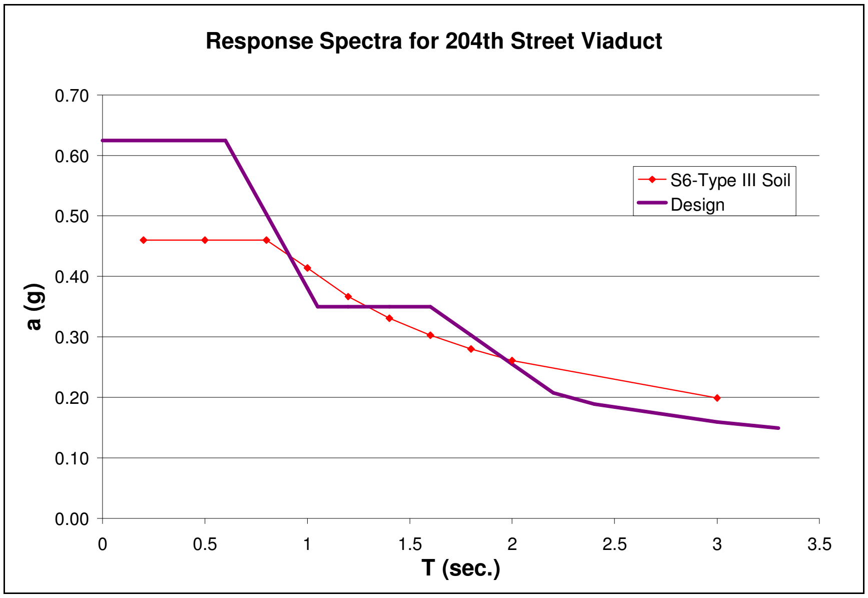


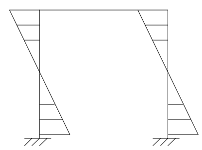
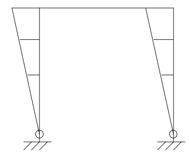
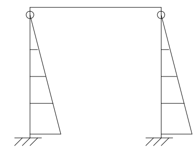
Figure 4. Design spectra for the 204th Street Viaduct project.

Framing Comparison

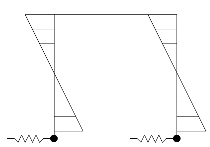
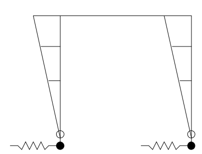
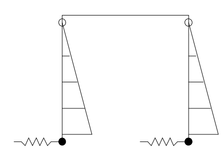
Some of the advantages and disadvantages of column pins compared to a rigid frame are summarized in Fig. 5. Fig. 5(a) compares the stiffness and base moment of each frame configuration for idealized conditions. As shown in the figure, the rigid frame is 4 times stiffer than the frame with pinned column tops. However, assuming an idealized spectra with a 1/T function the resulting base moments are equal.

Fig. 5(b) compares the three frames using actual parameters from the 204th Street viaduct, including lateral and rotational foundation springs. For simplicity the pins are still idealized at true hinges. With the soil springs, the stiffness of the rigid frame is only 1.5 times the stiffness of the frame with pinned column tops. Note that the frame with pins at the top engages the rotational spring, while the frame with pins at the base does not. Applying the site specific spectra, the pins at the top result in a base moment almost twice that of the rigid frame. One additional parameter not shown in the figure is that the frame with pins at the top has no additional axial load due to overturning, unlike the other two frames. This means the flexural capacity is the same for both columns and is constant.

This example illustrates the influence of actual conditions versus idealized conditions, and highlights the principal drawback of pinning the top of the columns, i.e., an increase in base moment. For the 204th St. project, however, the increased moment on the foundation could be accommodated at minimal cost with an increase in pile length.

	Rigid Frame	Pin at Column Base	Pin at Column Top
			
K	4K	K	K
T	½ T	T	T
a	2a	a	a
M	M	0	M

(a) Idealized Case

	Rigid Frame	Pin at Column Base	Pin at Column Top
			
K	10891 N/mm	10576 N/mm	7393 N/mm
T	2.27 sec	2.30 sec	2.76 sec
a	0.199	0.199	0.171
M	10.3 MN-m	0 MN-m	17.8 MN-m

(b) Actual Case (204th Street Viaduct)

Figure 5. Frame analysis for idealized case and actual case.

Pin Design

The column pins for the 204th St. viaduct are designed as concrete-filled tubes or CFT (also called CFST for concrete-filled steel tube). Under dead and live loads, the pins are subjected primarily to axial loading. With the superstructure continuous over the piers, the pins do not need to provide any bending moment resistance. They only need to accommodate pier cap rotations. Similarly, under earthquake loads the pins need sufficient axial load capacity, but no particular moment capacity: they need to accommodate rotations corresponding to the expected displacements. Moment capacity was checked for construction stages where uneven loading may occur before the girders are made continuous.

Design for Ultimate Limit States (ULS) checked the axial capacity of the column pins against the factored dead and live loads. Axial capacity was calculated using the composite column provisions of the Canadian Highway Bridge Design Code, CSA-S6-00 (CSA 2000). These are the same provisions found in CSA-S16.1 Limit States Design of Steel Structures (CSA 1994). Shams and Saadeghvaziri (1997) compare the provisions in ACI-318 Building Code Requirements for Reinforced Concrete (American Concrete Institute

1999) and AISC Specifications for Structural Steel Buildings (American Institute of Steel Construction 2005) with experimental results. Bruneau and Marson (2004) compare S16.1 and AISC provisions with experimental results. In both cases, the code provisions for axial capacity are found to be reasonable and conservative.

In addition to the ULS design, a Serviceability check was done to ensure adequate performance of the column pins under service conditions. Stresses due to axial loads were limited to $0.6^*\Phi f_y$, where Φf_y is the factored specified minimum yield stress of the steel tube. The axial stress is calculated using an equivalent transformed area for the concrete where the modulus of elasticity of the concrete is reduced to account for creep. The applied axial load is equal to the dead load plus 0.9 times the design live load, per CSA-S6-00 SLS Load Combination 1. This criteria ensures the stresses in both the steel tube and the concrete core are in the elastic range under service conditions. The diameter and wall thickness of the steel tubes were governed by the SLS criteria. Steel tube properties were summarized in Table 1 above. The thickest tubes were designed for a maximum axial load at service is 9.0MN and at ultimate is 11.8MN.

Seismic design of the column pins consists of ensuring sufficient rotational capacity to accommodate the anticipated lateral displacements of the superstructure. The response spectra analysis found a maximum elastic displacement at the superstructure level of 340 mm. A pushover analysis was used to confirm the columns and footings could maintain vertical load carrying capacity at 1.5 times the predicted elastic displacement. At that displacement, curvature demand on the pins is 0.15 rad/m using a 475 mm hinge length.

In tests, concrete-filled tubes show a very stable post-yield response under both monotonic and cyclic loading (Marson and Bruneau 2004, Shams and Saadeghvaziri 1997). In tests of columns subjected to axial and flexural loads, Class 1 steel tubes with relatively small diameter to thickness ratios show displacement ductility up to 7 before loss of moment capacity, and full hysteresis loops not noticeably affected by axial loads.

For the seismic design, the moment curvature capacity of the CFT pins was estimated using a simple axial strain compatibility analysis (local buckling is not included). The Mander et. al (1988) model was used for the confined concrete in the core and a generic steel model with strain hardening (Priestley et. al. 1996) was used for the steel tube. Recent research indicates the Mander model may overestimate confinement effects and flexural strength (Bruneau and Marson 2004), however, this does not appear to change available ductility. Fig. 6 shows the predicted moment curvature response for the thickest column pins. It indicates capacity well beyond the 0.15 rad/m curvature demand.

The column pins are designed with a 75 mm gap between the column and the pier cap and a debonded length of 200 mm each side of the gap. This provides a 475 mm unbonded length to distribute the curvature during seismic demands. The pins are embedded a minimum of one diameter beyond the debonded length (1.5 diameter from the surface), and are anchored with shear studs. A recent study of different embedment depths (Kingsley et. al., 2006) indicates full capacity of the steel tube can be mobilized with an embedment depth of 0.9 diameter.

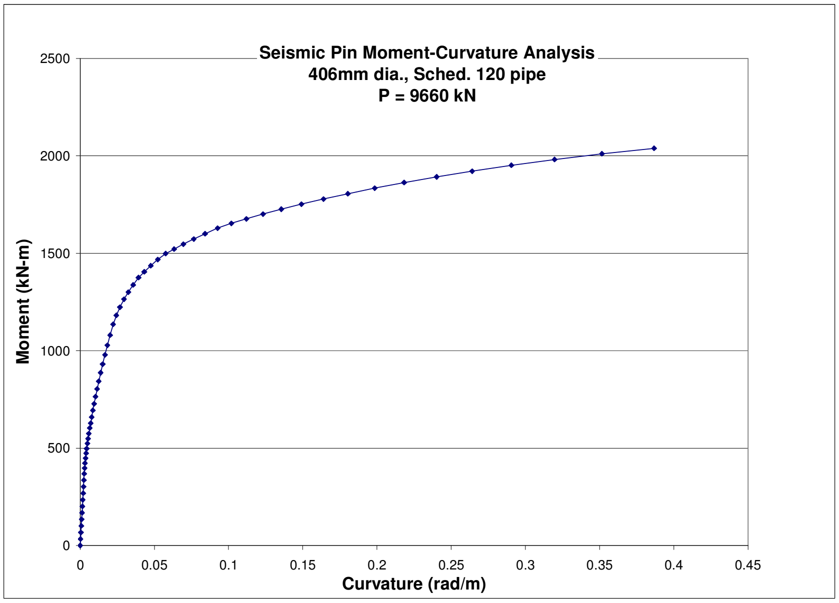


Figure 6. Predicted moment curvature response.

Construction

The 204th St. viaduct was a design-build project with a very tight schedule; construction had to start before design was completed. Design typically proceeds top-down, with some iteration between superstructure and substructure designs. Construction on the other hand proceeds bottom-up. By eliminating the transfer of any significant moment between the columns and pier caps, pins at the top of the columns enabled the design of the substructure to be completed ahead of the superstructure. Only a reasonable estimate of the superstructure weight is needed to complete the substructure and there is no need to iterate between the designs.

The simplified single-mode preliminary design was sufficient to determine demands on the footings and complete the design of the steel pipe piles very early in the project. This was important because ordering the piles was the first construction item on the critical path. Pile driving was able to start only 8 weeks after project award. The superstructure design, meanwhile, could be performed separately based entirely on dead and live loads. Only a final check for seismic loads was needed since the maximum moment transferred from the pins (1.6 to 2.0 MN) is small compared to the capacity of the pier caps (10 MN). This allowed the prestressed concrete girders to be designed and ordered quickly.

A full moment connection is easier to make between the column and the footing than between the column and the pier cap. Footings provide more concrete mass all around the columns and tend to have a relatively low reinforcement ratio. At the top of the column the pin is set within an inner spiral, which provides additional confinement and crack control (Fig.7). In the pier cap, reinforcement is provided in three directions to provide confinement and crack control around the pin, but with much less congestion than a full column connection (Fig.8). Post-tensioning of the pier caps reduces the need for shear reinforcement around the pins, further reducing congestion in the pier cap.

Figure 7. Pin inside column.

Figure 8. Pin zone reinforcement.

On the Sound Transit Link Light Rail Project, column pins were set into a pocket in the top of the column. The pins were then grouted in place after post-tensioning of the cap beams. This allowed deformations due to post-tensioning to occur freely instead of causing initial deformations in the columns. For the 204th Street project the post-tensioning was relatively light and this was not an issue. This points to a possible application of pins at the top of columns in a precast construction setting. Either the cap beams and or the columns can be precast with pockets left for the pins. The shear studs could be replaced with shear rings to further reduce the diameter of the required pocket.

Conclusions

Pinned columns increase the flexibility of the structure and reduce overall seismic demands. They can, however, significantly increase the bending moments at the fixed end of the column. Columns pinned at the top simplify the connection to the cap beam, while the full moment connection is easily accommodated in the footing.

The use of pins at top of columns allowed the design of the superstructure to be independent from the design of the substructure. This was extremely advantageous for the 204th Street Viaduct design-build project given the tight construction schedule.

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