Executive Summary

1. Introduction
Standard bridges are designed for collapse prevention under extreme seismic events based on ductile design concepts. Bridges with integral superstructures are common in high seismic regions. The superstructure and substructure are connected using rigid connections in these bridges to enhance the integrity of the bridge under seismic loading. The substructure may be constructed on an enlarged pile-shaft foundation in cases that the use of spread footing is not feasible. In a properly designed bridge, plastic hinges are formed in the columns to dissipate energy during strong earthquakes. The plastic hinges act as a fuse to prevent damage in the superstructure and foundations. Therefore, at least one end of the columns should be connected to the foundation or cap beam rigidly to force formation of the plastic hinges in the column. A hinge or a “pin” connection may be used at one end of the column to reduce the overall force demand leading to smaller and more economical foundations.

The economy of bridge projects can also be improved by reducing the construction time. The bridge construction process may disrupt flow of transportation, require long detour, or require costly use of temporary structures. Cast-in-place (CIP) is the prevailing bridge construction method, which requires concrete to set and cure onsite leading to slow construction. In contrast, prefabricating bridge components can reduce the construction time by eliminating the need for on-site curing and making use of components that have been fabricated in parallel. Another important factor is that the quality of construction would be enhanced by offsite fabrication due to a better control over the material quality and construction. Prefabrication of the elements facilitates accelerated bridge construction (ABC), which is rapid bridge construction using innovative planning, design, materials, and efficient construction methods in a safe and cost-effective manner.

2. Research Objectives
The main objectives of this study were to investigate the seismic performance of three types of bridge bent connections through experimental testing and analytical modeling:

1- Pipe-pin connections to develop hinge support at column-pile shaft joints for CIP and precast constructions.

2- Rebar-pin connections to develop hinge supports at column-pile shaft joint for CIP and precast constructions.

3- Pocket connections to develop rigid connections between precast columns and precast pier caps in bridges with integral superstructure-pier connections.

An additional objective of the study was to improve the current design guidelines for each connection type based on the results of the experimental, analytical studies, and parametric studies. An objective of the study was also to investigate the performance of ECC in column plastic hinge zones to mitigate damage in CIP and precast columns.
The study consisted of shake table testing of two large-scale bent specimens and performing comprehensive analytical studies of connections to determine the adequacy of the concepts developed in this and other studies and the need for further refinements.

3. Experimental Studies
The experimental part of the project was testing of two, two-column bents on shake tables of the Earthquake Engineering Laboratory at the University of Nevada, Reno. The test specimens were 1/3.75 scale models that were based on some of the features of the approach spans bents of the San Francisco Oakland Bay Bridge. Pin connections were provided in column to shaft connections in the test models. Two test models were built, BPSA and BRSA. BPSA stands for bent with pipe-pin column-pile-shaft connection for accelerated bridge construction (ABC) and BRSA stands for bent with rebar-pin column-pile-shaft connection for ABC. Pipe-pin connections were used in BPSA model and rebar-pins in BRSA model.

Two methods of construction for columns were implemented. South column in each bent was cast-in-place (CIP) and the other was precast (PC) to study different construction methods. The components on the south side of the bent are referred to as CIP (cast in place) components such as CIP column hereafter. The components on the north side of the bent are referred to as PC (precast) components such as PC column, hereafter. The proposed precast construction method consisted of two parts. The column shells were cast first. The pin was inserted in a 3-inch [76-mm] thick precast concrete shell of the column. Then, the shell was filled with SCC at the same time with the joint. The columns were connected to the precast bent cap with pocket connections. The protruded bars at the top end of the columns were inserted into the pocket in the precast cap beam. Finally, self-consolidating concrete (SCC) was placed in the hollow section (Fig. 1).

Pedestals were built under each column to model column to Type II pile-shaft connections. The pedestals were sufficiently tall to represent the structural behavior of pile-shafts. The study of soil interaction with pile-shafts was beyond the scope of this research. Details of the experimental studies and results are presented in Chapter 2 and 3 of this document, respectively.
3.1. Bent with Pipe-Pin Column-Pile Shaft Connection (BPSA)
The connections of the columns to pedestals were pipe-pins in this model. The joints between column and precast cap beam were pocket connections. Figure 2 shows the elevation view of BPSA. ECC was used over the full height of the plastic hinge region of the CIP column. The model was designed according to the Caltrans ductile design guidelines (Seismic Design Criteria, 2010). The connections were designed as capacity protected members with plastic hinging allowed only in the columns. While the columns were expected to undergo plastic deformation, the rest of the structure including the pipe-pins were designed to remain undamaged. Figures 3 and 4 show the details of the pipe-pin and pocket connections respectively.
Fig. 2. BPSA elevation view

Fig. 3. Pipe-pin details
The input ground motion needed to be strong enough to generate sufficient demand. Yet its displacement, velocity, and acceleration needed to be within the shake table limits. It was also preferred to have symmetric response to investigate the performance under full reversed cycles. The 142-degree of Sylmar Converter Station of the 1994 Northridge earthquake was chosen for simulation in the shake table tests (Fig. 5). The acceleration was filtered to have a symmetric ground motion. Additionally, the time axis of the acceleration record had to be shortened to account for the scale of the test model. The time axis was scaled by the square root of the geometric scale factor to account for the shorter period of the model relative to the prototype (Harris, 1982). The specimen was subjected to series of excitations until it reached the ultimate capacity of the bent. BPSA was tested under 11 motions with peak ground motion accelerations (PGAs) of 0.09g, 0.27g, 0.45g, 0.54g, 0.54g, 0.54g, 0.63g, 0.81g, and 0.99g, respectively. A white noise motion also applied to the specimen before each run and after the last run to capture the dynamic characteristics of the bent.
Detailed examination of the measured data indicated that the design of BPSA was satisfactory. Figure 6 shows the final condition of the specimen after the tests. Substantial plastic deformations were developed at the top of the columns (Fig. 7a and 7b). The lateral load capacity of the bent was reached due to plastic hinge failure in the CIP column, with no indication of damage in the pipe-pins. Minimal cracks were formed around the pipe-pin connection (Fig. 7c). The strain gages on the pipes also showed that the pipes remained elastic, as intended. Moments were developed at the bottom of the columns that were not considered in the design of the bent. The prefabricated column and pipe-pin performed as well as the cast-in-place elements. No difference was observed in the performance of the precast column and the cast in place column with pipe-pin connections. The pocket connections performed as designed to form the plastic hinge in the columns. Furthermore, the cap beam remained elastic. In addition, the column plastic hinge with ECC showed significantly less damage than that with conventional concrete (Fig. 7a and 7b).
Fig. 6. BPSA final condition after the tests

Fig. 7. Final conditions of elements in BPSA a) CIP column with ECC b) PC column c) CIP pipe-pin
Figure 8 shows the measured force-displacement hysteretic curves for BPSA. The first yield occurred in longitudinal bars of the PC column at displacement of -0.65 in [17 mm]. The displacement ductility of the bent was 3.6 from the envelope of the tests. However, the formation of full column plastic hinges and the strain data in plastic hinges indicate substantially higher ductility. The measured initial stiffness was highly reduced because of the slippage of the load cell to cap beam connection in the early runs (section 3.2.4). The force-displacement hysteresis curves of the bent indicated a slight pinching effect near the origin. The pinching is attributed to slippage at the pipe pins due to the closure of the gaps between the two pipes at each pin. The fluctuating pattern also indicates that friction release was gradual and did not occur at a specific displacement of the pipe-pin.

![Hysteresis Curves](image)

**Fig. 8. BPSA force-displacement relationship**

### 3.2. Bent with Rebar-Pin Column-Pile Shaft Connection (BRSA)

The connections of the columns to pedestals in this model were rebar-pins (Fig. 9). Similar to BPSA, the columns in BRSA were connected to the precast bent cap using pocket details. Additionally, ECC was used in the plastic hinge region of the precast concrete shell. The model was designed according to the Caltrans ductile design guidelines (Caltrans, 2010). The rebar-pins were designed based on previous research by Cheng et al. (2006). Figure 10 shows the detail of rebar-pins. They proposed a design method to provide sufficient ductility and shear capacity for the rebar-pins in a bent. While both rebar-pin and plastic hinge of the column undergo large plastic deformation, only the plastic hinge in the columns is expected to fail.
Fig. 9. BRSA elevation view

Fig. 10 Rebar-pin detail
The criteria of ground motion selection were similar to BPSA ground motion selection. Therefore, same input motion as of that of BRSA was used for BRSA (Fig. 5). It was tested under six motions with PGAs of 0.09g, 0.314g, 0.538g, 0.763g, 0.987g, and 1.211g, respectively. Before each run and after the last run a white noise motion applied to the bent. The white noise had frequency content of 0.7 to 40 Hz and amplitude of 0.05g similar to that in BPSA.

BRSA performed as it was designed. Figure 11 shows the final condition of BRSA after tests. Full plastic hinge capacity was reached at the top of the column while the rebar-pins did not fail. The rebar-pins underwent large plastic deformations under many cycles of earthquake loading without loss of capacity (Fig. 12a and 12b). The strains in the longitudinal reinforcement were safely under the ultimate strain. Despite the fact that the rebar-pin slipped horizontally, the shear friction was sufficient to resist the shear without gravitational axial load on the columns. The gap around the rebar-pin did not close in spite of the large rotations. The columns and pedestals did not yield near the rebar-pins but the rebar-pins and the surrounding concrete were damaged (Fig. 12c). The pocket connections of the columns to the cap beam performed as they were designed. The plastic hinges in the columns reached their rotation capacity while the reinforcement in the cap beam remained elastic. No damage was observed in the cap beam. Moreover, the PC column with ECC only in shell section showed significantly less damage than the CIP plastic hinge zone with conventional concrete (Fig. 12a and 12b).
4. Numerical Simulation of Experiments

Analytical studies of the test models were performed to validate the modeling assumptions based on the correlation between the analytical and experimental results. Three sets of analytical studies were conducted: pushover analysis using a simple stick model, pushover analysis using an elaborate model, and dynamic analysis of the shake-table tests. Details of the analytical studies and results are presented in Chapter 4 of this document.

4.1. Bent with Pipe-Pin Column-Pile Shaft Connection for ABC (BPSA)

4.1.1. Pushover Analysis Using Simple Stick Model

The forces and displacements of the bent were calculated using lumped plasticity. The simple stick model of BPSA consisted of two columns that were supported on pipe-pins and connected to a rigid cap beam at the top (Fig. 13a). The pipe-pins were modeled using the proposed pipe-pin springs. Linear rotational springs were used to model the moment-rotation relationship of the pipe-pins. The shear displacements of the pipe-pins were calculated assuming gap-rigid behavior. The calculated results are well correlated to the test data in terms of the stiffness, maximum base shear, and ultimate displacements with maximum error of 14% (Fig. 13b). The error was relatively small and acceptable considering the simple formulation of the analytical model.
4.1.2. Dynamic Numerical Simulations Using Frame Elements

Dynamic analysis of BPSA was performed in OpenSEES using frame and zero-length elements. The columns and pedestals were modeled using force-based (FB) elements, which are distributed plasticity elements. The columns to the pedestals connections were modeled using the proposed pipe-pin springs. The mass of the mass rig was added to the cap beam. The weight of the cap beam was also distributed on its nodes.

BPSA was analyzed using different material models, section discretization, and numbers of integration points to investigate the effects of modeling techniques on the analytical results. The results showed that the effects of these parameters on the calculated base shear and displacement are less than 10%. The calculated responses correlated well with the test results in terms of peaks, history shapes, and amplitudes. The calculated base shear and displacement histories for Run-10 are compared with the measured history in Fig. 14. The averages of the calculated peaks were within 20% of the measured data.
4.1.3. Quasi-Static Numerical Simulations Using Finite Element Modeling

Finite element (FE) models of BPSA were used for pushover analysis using ABAQUS/Explicit (V6.14-1) package. The purpose of the models was to investigate the complex interaction among different parts. Thus, the components of the pipe-pins were modeled with continuum elements to obtain more realistic force interaction through contact surfaces. A sketch of the finite element (FE) models is presented in Fig. 15. Those models were analyzed under quasi-static loading. The numerical simulations were verified by comparing the calculated and measured force-displacement envelopes.

Fig. 14. BPSA calculated and measured force and displacement histories for Run-10
The calculated and measured results are well correlated in terms of the yield points, ultimate displacements, and base shear capacity (Fig. 16). The maximum error subsequent to the elastic part of the curve was 15%. The numerical model overestimated the initial stiffness because the actual initial stiffness of the model was relatively low due to the cap beam deformations. In addition to the global force-displacement responses, the cracking patterns, pipe forces, and pipe-pin rotations were well correlated with the measured data (section 4.8.6). This model was subsequently used in parametric studies described in the next section.
The forces in the rods versus the bent lateral displacements are shown in Fig. 17. The yielding fore of the rod is shown using a dotted line. The rod forces were tensile even when the columns were under compression because of the rotation of the pipe. The rotations of the pipe-pins generated uplift at the base of the columns. The uplift generated relatively large elongations of the rod to compensate for the compression of the elastomeric pads. While the axial force in the rod was well below the yielding force, the stress condition needed to be checked for the combination of the axial force and flexure. The Von Mises yield stress was 75.1 ksi [518 MPa]. As Fig. 18 shows, the Von Mises stresses in the rod reached the yielding criteria. The fixity of the rod at the end plates produced significant moments under the movements of the pipes and yielded the rod under flexure (dark regions on the rods in Fig. 18).

Fig. 16. Comparison of results of FE analysis with the tests envelopes
Fig. 17. BPSA threaded rods axial force

Fig. 18. Von Mises stress (psi) in the rods, South direction: (a) CIP (b) PC, North direction: (c) CIP (d) PC
4.2. Bent with Rebar-Pin Column-Shaft Connection for ABC (BRSA)

4.2.1. Static Numerical Simulations Using Simple Stick Model
A simple stick model of BRSA using the lumped plasticity springs was developed as a design tool. That model was composed of two elastic-plastic column elements supported on rebar-pins and connected to a rigid cap beam at top (Fig. 19a). The Paulay and Priestley (1992) plastic hinge model was used in the numerical simulation of the columns. Furthermore, the rotational behavior of the rebar-pins was modeled using bilinear rotational springs based on the Cheng et al. (2006) model. Moreover, the slippage of the rebar-pins was estimated from the average of the crack shear-slip models. Figure 19b compares the measured and calculated force-displacement relationships. The calculated displacement ductility was 7.4, which is 5% larger than the measured ductility. Despite the simplifying assumptions, the calculated and measured data were reasonably close.

![Fig. 19](image)

(a) Pushover analysis of BRSA using simple stick model
(b) calculated and measured force-displacement relationships

4.2.2. Static Numerical Simulations Using Frame Elements
Pushover analysis of BRSA was performed in OpenSEES using frame and spring elements. The pushover response was estimated twice, once using the lumped (springs) and again the distributed plasticity models of the rebar-pins (Fig. 20a and 21a). The spring properties of the rebar-pins were based on the Cheng et al. (2006) model. For the distributed model of rebar-pins, displacement-based (DB) and force-based (FB) elements were used. In both models, the columns and pedestals were modeled using the FB elements. The cap beam was modeled using an elastic element. The failure criteria in the analyses were the bent lateral displacement at which either the core reached its ultimate strain capacity or reinforcing bars fractured. As Figs. 20b and 21b show, the correlation of the results from both models with the test results is acceptable in terms of initial stiffness, yield displacement, ultimate base shear, and ultimate
displacement. The model with the springs for the rebar-pins was less sensitive to the displacement increments and number of integration points. The model with spring model of rebar-pins was subsequently used in parametric studies described in the next section.

![Diagram](image)

(a) Fig. 20. BRSA pushover analyses with lumped plasticity for rebar-pins a) bent model b) calculated and measured force-displacement relationships

![Diagram](image)

(a) Fig. 21. BRSA pushover analyses with distributed plasticity for rebar-pins a) bent model b) calculated and measured force-displacement relationships

4.2.3. Dynamic Response Simulation Using Frame Elements
Similar numerical models to the pushover analyses were adopted to analyze BRSA under dynamic loading to simulate the shake table tests (Figs. 20a and 21a). The calculated base shear and displacement using the spring model of the rebar-pins correlated well with the test results in terms of peaks, waveforms, and the base shear and displacement amplitudes. The calculated base shear and displacement histories for Run-4 are compared with the measured history in Fig.
Alternatively, the responses were calculated using the distributed plasticity model of the rebar-pins. Despite the good correlation in the base shear, the displacements were underestimated by as much as 26% using the distributed plasticity model of rebar-pins.

Fig. 22. BPSA calculated and measured force and displacement histories for Run-4

5. Parametric Studies
The effects of key parameters on the performance and capacity need to be investigated to develop a general design procedure for column-pile-shaft pin connections. For that reason, parametric studies were performed to fill in the knowledge gap that was not covered in the experimental and analytical studies. Details of the parametric analyses and results are presented in Chapter 5 of this document.

5.1. Pipe-Pin Connections of Column to Pile-Shaft
The influence of axial load index, lower pipe taper slope, strands instead of rods, and removing the tension member on the bent response and pipe-pin performance were studied. A summary of the general findings is listed below:

1. The pipe-pin moment significantly increased the base shear. Therefore, inclusion of the base moments is necessary for a safe design.
2. Increase of axial force increases the pipe-pin moments. Therefore, the axial load ratio should be included in the calculations of the pipe-pin moments to calculate the base shear.
3. The increase of axial force delays the development of the extension in the tension members and reduces the stress in those members. However, the rod remains in tension under large pipe-pin rotations.
4- The large column axial force produces compressive force in the rod. The development of this force should be prevented by isolating the nuts and rod from concrete.
5- The tapering of the lower pipe improves the behavior of the pipe-pin insignificantly. Therefore, a conical surface for the lower pipe is not necessary.
6- Using strands instead of rods reduces the base shear. The strands remain elastic under large rotations because they are not subjected to flexure. Therefore, it is suggested to use posttensioning strands as the tension members in pipe-pins.

5.2. Rebar-Pin Connections of Column to Pile-Shaft
The influence of the axial load index and core diameter on the bent response and pin performance were studied. A summary of the general findings is listed below:

1. The rebar-pin moment should be included in calculation of the base shear.
2. The axial force needs to be less than 20% of the rebar-pin ultimate compressive capacity to avoid strength loss.
3. An increase of axial force improves the safety factor against shear friction failure of the rebar-pins.
4. The increase of the axial load improves the flexural safety factor as long as the mode of failure of the rebar pin is not dominated by compressive failure of the concrete in the pin.
5. The reduction of the core diameter increases the ductility and reduces the pin moment. However, the core diameters smaller than 50% of the section cause softening in the force-displacement of the bent and are not desirable.

6. Design procedure
Pipe-pin and rebar-pin connections of column to pile-shaft are designed to transfer column shear and axial force while minimizing the moment transfer between the column and the pile-shaft. The findings of the present study were combined with those from previous studies and design codes to develop a practical design procedure. The details of the proposed design methods for “pinned” connections of column to pile-shaft are presented in Chapter 6 in this document.

6.1. Pipe-Pin Connection of Columns to Pile-Shafts
The pipes transfer column shear to the pile. A tension member within the pipes transfers the column uplift force. A pad at the interface of the column and pile-shaft provides rotational capacity and transfers the compressive column axial force. The experimental and analytical studies showed that the interaction of the forces in the column pile-shaft connection produced some moment at the connection.

6.1.1. Design Force and Moment Demand
Pipe-pins are designed as capacity-protected members to remain undamaged during large earthquakes. Therefore, pipe-pins are designed to resist the forces generated when the structure reaches its collapse limit state (CLS).

Pipe-Pin Moments: The pipe-pin moment, $M_{u,\text{pin}}$, is developed by the compressive force in the pad and the contact force between the pipes (Fig. 23). It was concluded based on analytical studies that the moment due to pipe contact was less than 10% of the total pipe-pin moment. Therefore, it was not included in the calculation of pin moment. A linear rotational spring was
proposed to calculate the pipe-pin moments. The pipe-pin moment at the column base is estimated by the following equations.

\[ M_{u,\text{pin}} = \frac{M_g}{\theta_g} \times \theta_{u,\text{pin}} \]  
\[ \theta_g = \frac{2 t_{pad}}{D_{col}} \]  
\[ M_g = \frac{K_{c,\text{pad}}}{K_{c,\text{pad}} + K_{tm}} \left( \frac{OD_{pad}}{2} - K_{tm} \theta_g + P \right) \times \frac{OD_{pad}}{2} \]  
\[ K_{tm} = \frac{E_{tm} \times A_{tm}}{L_{tm}} \]  
\[ K_{c,\text{pad}} = \frac{E_{pad} \times A_{pad}}{t_{pad}} \]

where,

- \( A_{tm} \): cross-sectional area of tension member (center rod or tendon), in\(^2\) [mm\(^2\)]
- \( A_{pad} \): plan view area of the pad, in\(^2\) [mm\(^2\)]
- \( D_{col} \): column diameter, in [mm]
- \( E_{pad} \): modulus of elasticity of the pad, ksi [MPa]
- \( E_{tm} \): modulus of elasticity of the tension member, ksi [MPa]
- \( K_{c,\text{pad}} \): compressive stiffness of the pad, kip/in [kN/mm]
- \( K_{tm} \): axial stiffness of the tension member, kip/in [kN/mm]
- \( L_{tm} \): effective length of the tension member, center-to-center of the nuts (Fig. 6-3), in [mm]
- \( M_g \): moment to close the gap
- \( M_{u,\text{pin}} \): base moment at the pipe-pin, kip.in [kN.m]
- \( OD_{pad} \): outer diameter of the pad, in [mm]
- \( P \): column axial force, positive sign for compression, kip [kN]
- \( \theta_g \): rotation to close the gap, rad
- \( \theta_{u,\text{pin}} \): ultimate base rotation, estimated equal to the drift ratio, rad
- \( t_{pad} \): pad thickness, in [mm]
Column Shear: The shear demand on the column and adjacent members are associated with the overstrength column moment. Consequently, the overstrength base shear in the bent, $V_{u,bent}$, is the summation of the column shears as follows.

$$M_{o,\text{col}} = 1.2M_{p,\text{col}}$$

$$V_{u,\text{col}} = \frac{M_{o,\text{col}} + M_{u,\text{pin}}}{H_c}$$

$$V_{u,bent} = \sum V_{u,\text{col}}$$

where,

- $H_c$: column clear height, in [mm]
- $M_{o,\text{col}}$: column overstrength moment, kip.in [kN.m]
- $M_{u,\text{pin}}$: pipe-pin base moment, kip.in [kN.m]
- $V_{u,\text{col}}$: column shear demand, kip [kN]
**Column Axial Force:** The dead load generates equal axial forces, $P_{dl}$, in the columns of a symmetric bent. The overturning moment, $OM$, redistributes the axial force in the columns by increasing the axial force in one column and decreasing it in the other column. In the cases that the overturning moment is larger than the dead load moment, an uplift force is generated in the column.

$$OM = V_{u,bent} \times H - 2M_{u,pin}$$

$$T_{u,col} = P_{dl} - \frac{OM}{S}$$

$$P_{u,col} = P_{dl} + \frac{OM}{S}$$

**Tension Member Force:** The axial force in the pin tension member is calculated according to the following equation.

$$T_{u,tm} = \frac{K_{tm}}{K_{c,pad} + K_{tm}} \left( \frac{OD_{pad}}{2} K_{c,pad} \theta_{u,pin} - P \right)$$

**Threaded Rod Moment Demand:** As the upper pipe tends to move laterally and rotates, the partial fixity of the rod-end plate connection produces moment in the rod. The rod moment demand is calculated according to the recommendations by Mehrsoroush and Saiidi (2014), assuming that the upper pipe rotates as a rigid body.

$$M_{u,rod} = \frac{3 E_{rod} I_{rod}}{L_{rod}^2} \delta_{rod}$$

$$\delta_{rod} = \theta_{u,pin} H_{upper}$$

**Impact Force between the Pipes:** The upper pipe impacts the lower pipe subsequent to the friction release. The impact force is calculated from the following equation.

$$F_{impact} = 0.9 \times \sqrt{\frac{P_{dl} G_{h} E I_{cr, col}}{H_{c}^3}}$$

**Lower Pipe and Upper Pipe Shear Demand:** The shear demand in the pipes is the summation of the column shear and the impact force.

$$V_{u,pipes} = V_{u,col} + F_{impact}$$

6.1.2. Design Capacities

**Lower Pipe:** The lower pipe is designed to resist the column shear and the impact force according to the following equation based on the recommendations by Zaghi and Saiidi (2010).

$$V_{u,pipes} = V_{u} + F_{impact} \leq \phi V_{n,lower}$$
\[ V_{n,lower} = \min \left( V_{n,lower}^{pipe}, V_{n,lower}^{Pile} \right) \]

\[ V_{n,lower}^{pipe} = \frac{F_{y,pipe}}{\sqrt{3}} A_{pipe} \]

\[ L_1 = \sqrt{e^2 + \frac{2M_p}{OD_{lower} f_c^*}} - e \]

\[ e = L_{protrude} \]

\[ H_u = L_1 OD_{lower} f_c^* \]

\[ V_{n,lower}^{lower \ pipe} = F_{y,pipe} \sqrt{3} A_{pipe} \]

\[ L_1 = \sqrt{\frac{2M_p}{OD_{lower} f_c^*}} - L_{protrude} \]

\[ H_u = L_1 OD_{lower} f_c^* \]

\[ V_{n,lower}^{noAxial} = \left( \sqrt{\frac{L_{protrude}^2}{2} + \frac{2M_p}{OD_{lower} \times f_{c}^*} - L_{protrude}} \right) \times OD_{lower} f_{c}^* \]

\[ P_{max,pile} = \begin{cases} 
1A'_{c,lower} \text{ (ksi)} \\
0.007A'_{c,lower} \text{ (MPa)} 
\end{cases} \]

For circular pile-shafts: \( A'_{c,lower} = \frac{2\pi - 2\alpha_i + \sin(2\alpha_i)}{2} \left( \frac{D_{pile}}{2} \right)^2 - \pi OD_{lower}^2 \frac{4}{4} \)

\[ \alpha_i = \arccos \left( \frac{D_{pad}}{D_{pile}} \right) \text{ (radians)} \]

\[ V_{c,lower}^{lower} = 0.8 A'_{c,lower} f_{c}^{'} \tan(54^\circ) \]

\[ f_{c}^{'} = \begin{cases} 
0.142 \sqrt{f_{c}^*} \text{ (ksi)} \\
0.374 \sqrt{f_{c}^*} \text{ (MPa)} 
\end{cases} \]

\[ V_{p,lower}^{lower} = \frac{M_p}{D_{pad} + OD_{lower} \tan(54^\circ)} = \frac{1.45 M_p}{D_{pad} + OD_{lower}} \]

\[ V_{p,lower}^{sp} = 0.34 \frac{A_{sp} f_{yz} d_{pile} (\cos(\alpha_i) \sin(\alpha_i) + \pi - \alpha_i)}{s_{pile}} \]  \hspace{1cm} (Zaghi and Saiidi, 2010)

\[ V_{n,lower}^{maxAxial}^{lower} = V_{c,lower}^{lower} + V_{p,lower}^{lower} + V_{sp,lower}^{lower} \]

\[ V_{n,lower}^{lower} = V_{n,lower}^{noAxial} + \left( V_{n,lower}^{maxAxial} - V_{n,lower}^{noAxial} \right) \left( \frac{P_u}{P_{max,pile}} \right)^{0.7} \]

where,

\( V_{n,lower}^{lower} \) : nominal lower pipe shear strength, kip [kN]

\( \phi \) : strength reduction factor, which is 0.75 according to Zaghi and Saiidi (2010) recommendation

\( V_{n,lower}^{pipe} \) : nominal lower pipe shear capacity, kip [kN]

\( V_{n,lower}^{pile} \) : nominal cracking shear capacity of pile-shaft, kip [kN]
\( F_{y,\text{pipe}} \): pipe yield strength, ksi [MPa]
\( A_{\text{pipe}} \): pipe gross section area, in\(^2\) [mm\(^2\)]
\( L_1 \): depth of pipe plastic hinge from the pile surface
\( f_c^* \): concrete bearing stress, ksi [MPa]
\( M_p \): pipe plastic moment, in\(^2\) [mm\(^2\)]
\( e \): eccentricity of the pipes contact point from pile-shaft surface, in [mm]
\( OD_{\text{lower}} \): lower pipe outer diameter, in [mm]
\( l_{\text{protrude}} \): distance from the top of the lower pipe to the top of the pile-shaft, in [mm]
\( Z_{\text{pipe}} \): plastic section modulus, in\(^3\) [mm\(^3\)]
\( A_c' \): horizontal projection of the cracking plane, in\(^2\) [mm\(^2\)]
\( D_{\text{pile}} \): pile-shaft diameter, in [mm]
\( D_{\text{pad}} \): outer diameter of bearing pad, in [mm]
\( OD_{\text{lower}} \): outer diameter of lower pipe, in [mm]
\( M_p \): lower pipe plastic moment, kip.in [kN.m]
\( A_{\text{sp}} \): spiral sectional area in pile-shaft, in\(^2\) [mm\(^2\)]
\( d_{c\text{pile}} \): center-to-center diameter of pile-shaft spiral, in [mm]
\( f_{ys} \): yield strength of spirals, ksi [MPa]
\( s_{\text{pile}} \): pitch of spiral, in [mm]

**Upper Pipe:** The upper pipe is designed by the method proposed by Mehrsoroush and Saiidi (2014) in three steps:

\[
V_{u,\text{pipes}} \leq \phi V_{n,\text{upper}} \quad (33)
\]

\[
V_{n,\text{upper}} = V_{n,\text{upper}}^{\text{noAxial}} + (V_{\text{maxAxial}} - V_{n,\text{upper}}^{\text{noAxial}}) \left( \frac{P_u}{P_{\text{max,}\text{col}}} \right)^{0.7} \quad (34)
\]

\[
P_{\text{max,}\text{col}} = \begin{cases} 
1A_{c,\text{upper}}' (\text{ksi}) \\
0.007A_{c,\text{upper}}' (\text{MPa})
\end{cases} 
\quad (35)
\]

\[
V_{n,\text{upper}}^{\text{noAxial}} = V_c + V_{sp} = f_c^* OD_{\text{upper}} L_2 \quad (36)
\]

\[
L_2 = \sqrt{e_2^2 + \frac{2M_p}{OD_{\text{upper}} f_c^* + e_2}} \quad (37)
\]

\[
e_2 = L_{\text{protrude}} - t_{\text{pad}} \quad (38)
\]

\[
M_p = Z_{\text{pipe}} F_{y,\text{pipe}} \quad (39)
\]

\[
V_c = 0.8 f_c A_c + f_c (D_{\text{col}} - OD_{\text{upper}}) L_2 \quad (40)
\]

\[
A_c = \frac{\pi (D_{\text{col}}^2 - OD_{\text{upper}})}{8} \quad (41)
\]
\[ f_v = \begin{cases} 0.095 \sqrt{f_c'} \text{ (ksi)} \\ 0.25 \sqrt{f_c'} \text{ (MPa)} \end{cases} \] (42)

\[ f_t = \begin{cases} 0.24 \sqrt{f_c'} \text{ (ksi)} \\ 0.62 \sqrt{f_c'} \text{ (MPa)} \end{cases} \] (43)

\[ V_{sp} = \frac{1}{4} \left( \frac{f_{ys} A_{sp} \pi d_{col}}{4 s_{col}} + f_{ys} A_{sp} 2L_2 \right) \] (44)

\[ f_{c'}^* = \frac{V_c + V_{sp}}{OD_{upper} \times L_2} \leq \begin{cases} \sqrt{f_c'} \frac{2.95 - \frac{3}{3.35} \sqrt{OD_{upper}}}{2.43} f_{c'}' \\ \sqrt{f_c'} \frac{2.95 - \frac{3}{9.85} \sqrt{OD_{upper}}}{6.38} f_{c'}' \end{cases} \] (45)

\[ V'_{c,upper} = 0.8 A_{c'} f_{v'} \tan(54°) \] (46)

\[ f_{v'} = \begin{cases} 0.142 \sqrt{f_c'} \text{ (ksi)} \\ 0.374 \sqrt{f_c'} \text{ (MPa)} \end{cases} \] (47)

\[ A_{c'} = \frac{2\pi - 2\alpha_u + \sin(2\alpha_u)}{2} \left( \frac{D_{col}}{2} \right)^2 - \frac{\pi OD_{upper}^2}{4} \] (48)

\[ \alpha_u = \arccos \left( \frac{D_{pad}}{D_{col}} \right) \text{ (radian)} \] (49)

\[ V'_{p,upper} = \frac{1.45 M_p}{D_{pad} - 1.45 e_2} \] (50)

\[ V'_{sp,upper} = \frac{0.34 A_{sp} f_{ys} d_{col} \left( \cos(\alpha_u) \sin(\alpha_u) + \pi - \alpha_u \right)}{s_{col}} \] (51)

\[ V'_{n,upper} = V'_{c,upper} + V'_{p,upper} + V'_{sp,upper} \] (52)

where,

- \( L_2 \): depth of pipe plastic hinge in the column from the bottom of column
- \( f_{c'} \): equivalent concrete bearing stress, ksi [MPa]
- \( M_p \): pipe plastic moment using, in² [mm²]
- \( e_2 \): eccentricity inside the pipe, in [mm]
- \( l_{protruded} \): the distance from the top of the lower pipe to the bottom of column, in [mm]
- \( OD_{upper} \): upper pipe outer diameter, in [mm]
- \( A_{c'} \): horizontal projected area of cracked section, in²
- \( f_v \): lower bound concrete shear strength, ksi [MPa]
- \( f_t \): lower bound concrete tensile strength, ksi [MPa]
- \( D_{col} \): column diameter, in [mm]
- \( OD_{upper} \): upper pipe outer diameter, in [mm]
**Threaded Rod:** The threaded rod is designed for the combination of the tensile force and bending.

\[
\text{if } \frac{T_{u,\text{rod}}}{T_{r,tm}} < 0.2 \text{ then, } \frac{T_{u,\text{rod}}}{2T_{r,\text{rod}}} + \frac{M_{u,\text{rod}}}{M_{r,\text{rod}}} \leq 1.0
\]  
(53)

\[
\text{if } \frac{T_{u,\text{rod}}}{T_{r,tm}} \geq 0.2 \text{ then, } \frac{T_{u,\text{rod}}}{T_{r,\text{rod}}} + \frac{8M_{u,\text{rod}}}{9M_{r,\text{rod}}} \leq 1.0
\]  
(54)

\[M_{r,\text{rod}} = \phi_f M_{n,\text{rod}}
\]  
(55)

\[M_{n,\text{rod}} = \min(M_{p,\text{rod}}, 1.6M_{y,\text{rod}})
\]  
(56)

\[M_{p,\text{rod}} = F_{y,\text{rod}} Z_{\text{rod}}
\]  
(57)

\[M_{y,\text{rod}} = F_{y} S_{\text{rod}}
\]  
(58)

\[Z_{\text{rod}} = \frac{D_{\text{rod}}^3}{6}
\]  
(59)

\[S_{\text{rod}} = \frac{\pi D_{\text{rod}}^3}{32}
\]  
(60)

\[T_{r,tm} = \min(\phi_f F_{y,\text{tm}} A_{g,\text{tm}}, \phi_u F_{u,\text{tm}} A_{g,\text{tm}})
\]  
(61)

where,

- \(T_{u,\text{rod}}\): factored tensile demand of threaded rod, kip [kN]
- \(T_{r,tm}\): factored tensile capacity of threaded rod, kip [kN]
- \(M_{u,\text{rod}}\): moment demand on threaded rod, kip.in [kN.mm]
- \(M_{r,\text{rod}}\): factored flexural capacity of threaded rod, kip.in [kN.mm]
- \(M_{n,\text{rod}}\): nominal flexural capacity of threaded rod, kip.in [kN.mm]
- \(M_{p,\text{rod}}\): plastic moment of threaded rod, kip.in [kN.mm]
- \(M_{y,\text{rod}}\): yield moment of threaded rod, kip.in [kN.mm]
- \(Z_{\text{rod}}\): plastic section modulus of threaded rod, in³ [mm³]
\( S_{rod} \): elastic section modulus of threaded rod, in\(^3\) [mm\(^3\)]
\( D_{rod} \): threaded rod nominal diameter, in [mm]
\( \phi_f \): strength reduction factor for flexure, which is 1.0
\( \phi_y \): strength reduction factor for yielding of tension member, which is 0.95
\( \phi_u \): strength reduction factor for fracture of tension member, which is 0.80

**Strands:** Strands are designed only for tension using the following equations.

\[
T_{u,tm} < T_{u,strand} \quad (62)
\]
\[
T_{r,tm} = \min(\phi_y F_y A_{g,tm}, \phi_u F_u A_{g,tm}) \quad (63)
\]

where,

\( T_{u,strand} \): factored tensile demand of strand, kip [kN]

**Studs:** The studs are designed to transfer the entire tensile capacity of the tension member to concrete through shear in the studs welded on the pipes.

\[
V_{r,stud} = \phi_{v,stud} f_u A_v \leq \frac{T_{r,tm}}{n_{studs}} \quad (64)
\]

where,

\( V_{r,stud} \): shear capacity of one stud, kip [kN]
\( \phi_{v,stud} \): strength reduction factor for shear of studs, which is 0.65
\( f_u \): specified tensile strength of shear studs, ksi [MPa]
\( A_v \): cross-sectional area of shear stud, in\(^2\) [mm\(^2\)]
\( n_{studs} \): number of studs on each pipe
\( T_{r,rod} \): factored tensile capacity of threaded rod or strands, kip [kN]

6.1.3. Detailing Recommendations
Based on the results of the parametric studies and previous studies, a series of detailing recommendations for design of the pipe-pin connections of column to pile-shaft were presented in section 6.2.4 of this document.

6.1.4. Design Steps
In summary, the following steps are proposed to design pipe-pin connections between columns and pile-shafts:

**Step 1.** Determine the force and moment demands assuming base moment is zero.

**Step 2.** Determine the pad thickness based on the maximum expected pin rotation.

**Step 3.** Determine the dimension of the pipes.

**Step 4.** Determine the shear demand on the pipes.

**Step 5.** Check the lower pipe shear strength and adjust the dimension as necessary.
Step 6. Proportion the rubber pad based on the lower pipe dimensions.

Step 7. Design the tension member.

Step 8. Recalculate the force and moment demands including the pipe-pin moment.

Step 9. Repeat step 5 to 8 until the base moment converges.

Step 10. Design the upper pipe thickness based on the shear demand.

Step 11. Design the studs.

6.2. Rebar-Pin Connection of Columns to Pile-Shafts
Rebar-pins are designed to transfer shear and axial force while undergoing plastic deformations under strong earthquakes. The hinge longitudinal reinforcement is expected to yield and the cover concrete is expected to be damaged. The experimental and analytical studies of the present study showed that the Cheng et al. (2006) design provisions are generally adequate to design column to pile-shaft connections, and hence those provisions were adopted with necessary refinements to make them applicable to column-pile-shaft connections.

6.2.1. Design Force and Rotation Demand
The force demands on the rebar-pin connection are calculated using CLS, which is the global collapse mechanism with plastic hinges at the top of the columns and the rebar-pins.

Rebar-Pin Plastic Moment: The rebar-pin plastic moment, \( M_{pp\text{pin}} \), is calculated based on the moment-curvature analysis of the hinge section assuming that the section was doubly confined, (1) provided by the transverse steel in the hinge, and (2) provided by the confined concrete in the column and pile-shaft immediately adjacent to the hinge (Cheng et al., 2006). The hinge cover concrete properties were modified using the average of confinement pressures generated by the column and pedestal transverse reinforcement. Those confinement pressures were added to the confinement pressures from the hinge transverse steel to determine the confinement pressure of the core concrete in the rebar-pins.

The plastic moment in the rebar pin is determined by bilinear or quadrilinear idealization of the moment-curvature relationship. It is proposed to idealize the moment-curvature relationship by a quadrilinear curve according to section 5.3.3 if the plastic moment using bilinear idealization is less than 90% of the maximum moment. Using either idealization methods, the rebar-pin plastic moment would be the maximum moment in the idealized moment-curvature relationship. Alternatively, the rebar-pin plastic moment is estimated conservatively as the maximum moment from the moment-curvature relationship prior to idealization.

Column and Rebar-Pin Shear: The column and rebar-pin shear demands are associated with the overstrength column and rebar-pin moment. The overstrength moments and shear demand, \( V_{s,\text{col}} \), are calculated using the following relationships.

\[
M_{o,\text{col}} = 1.2M_{p,\text{col}} \tag{65}
\]

\[
M_{o,\text{pin}} = 1.2M_{p,\text{pin}} \tag{66}
\]
\[ V_{u,\text{col}} = \frac{M_{o,cot} + M_{o,pin}}{H_c} \] (67)
\[ V_{u,\text{base}} = \sum V_{u,\text{col}} \] (68)

where,
\( H_c \): column clear height, in [mm]
\( M_{o,cot} \): column overstrength moment, kip.in [kN.m]
\( M_{o,pin} \): rebar-pin overstrength moment, kip.in [kN.m]
\( V_{u,\text{col}} \): column and rebar-pin shear demand, kip [kN]
\( V_{u,\text{bent}} \): overstrength base shear in bent, kip [kN]

6.2.2. Design Capacities

**Rebar-Pin Shear Capacity:** Based on the Cheng et al. (2006) study, the rebar-pin shear capacity is the same as the friction capacity of the hinge. The friction capacity is determined using a coefficient of friction of 0.45 and a clamping force that is the total compressive force in the section obtained from moment-curvature analysis. The friction coefficient accounts for loss of friction due to the cyclic action of earthquake forces in the hinge.

\[ \phi V_{n,pin} > V_{u,\text{col}} \] (69)
\[ V_{n,pin} = \mu C \] (70)
\[ C = C_c + C_s = P + T_s \] (71)

where
\( V_{u,\text{col}} \): column and rebar-pin shear demand, kip [kN]
\( V_{n,pin} \): rebar-pin shear capacity, kip [kN]
\( C \): total compressive force in the hinge section from moment-curvature analysis, kip [kN]
\( C_c \): compressive force in concrete from moment-curvature analysis of hinge section, kip [kN]
\( C_s \): compressive force in steel from moment-curvature analysis of hinge section, kip [kN]
\( P \): column axial force, with compressive force being positive, kip [kN]
\( T_s \): tensile force in steel from moment-curvature analysis of hinge section, kip [kN]
\( \mu \): coefficient of friction, which is 0.45
\( \phi \): strength reduction factor, which is 0.85

**Rebar-Pin Rotation Capacity:** Based on the Cheng et al. (2006) model, the rebar-pin ultimate rotation capacity is calculated by assuming that the plastic deformations occur over an equivalent plastic hinge length at the rebar-pin. The gap thickness should be sufficiently large to accommodate the rotation capacity of the rebar-pin.

\[ \theta_{n,pin} < \theta_{close} \] (72)
\[ \theta_{close} = \arcsin\left( \frac{g}{0.5D_{col}} \right) \]  

\[ \theta_{n,pin} = \theta_p + \theta_e \]  

\[ \theta_e = g \times \phi_{y,\text{pin}} \]  

\[ \theta_p = \phi_{p,\text{pin}} \times L_{p,\text{pin}} \]  

\[ \phi_{p,\text{pin}} = \phi_{u,\text{pin}} - \phi_{y,\text{pin}} \]  

\[ L_{p,\text{pin}} = g + 0.15 \times f_y \times d_p (\text{in}, ksi) \]  

\[ = g + 0.022 \times f_y \times d_p (mm, MPa) \]

where,

\[ \theta_n: \] rebar-pin rotation capacity, rad

\[ \theta_e: \] rebar-pin elastic rotation, rad

\[ \theta_p: \] rebar-pin plastic rotation, rad

\[ \phi_{p,\text{pin}}: \] rebar-pin plastic curvature, in\(^2\) [mm\(^{-1}\)]

\[ \phi_{u,\text{pin}}: \] ultimate curvature of rebar-pin from moment-curvature analysis, in\(^2\) [mm\(^{-1}\)]

\[ L_{p,\text{pin}}: \] equivalent plastic hinge length of rebar-pin, in

\[ f_y: \] rebar-pin longitudinal bar yield strength, ksi [MPa]

\[ d_p: \] diameter of longitudinal bars in rebar-pin, in

\[ g: \] gap thickness, in

6.2.3. Detailing Recommendations

Based on the results of the parametric studies and previous studies, a series of detailing recommendations for design of the rebar-pin connections of column to pile-shaft were presented in section 6.3.4.4 of this document.

6.2.4. Design Steps

**Step 1.** Determine the rebar-pin dimension, core diameter, and the required longitudinal steel.

**Step 2.** Calculate confined concrete properties for the hinge cover concrete. The cover concrete is confined by the average of column and pile-shaft confinement pressure.

**Step 3.** Determine the hinge transverse reinforcement for target curvature ductility of 10 using Mortensen and Saiidi (2002) performance based design method. The hinge core concrete properties are calculated based on the double confinement from the hinge transverse steel and confinement steel in the column and the pile-shaft adjacent to the rebar-pin.

**Step 4.** Determine moment-curvature relationship for the rebar-pin and column.

**Step 5.** Determine demand forces and rotation.

**Step 6.** Check rebar-pin friction capacity. If the capacity is not sufficient, adjust the reinforcement or size of the hinge, and repeat steps 4 to 7.
Step 8. Check hinge gap closure and determine hinge gap thickness to prevent gap closure.

Step 9. Determine the reinforcement detailing for the rebar-pin.

7. Observations
Noteworthy observations made in the course of the experimental and analytical studies presented in this document were:

1- In both specimens, full plastic hinge capacity was reached at the top of the column while the pins did not fail.
2- Moments were developed at both pin types leading to an increase in the base shear by approximately 30%. Even the pipe-pins without any tension members and the rebar-pins with very small core diameters generated significant moments at the column base.
3- The damage in the pipe-pin connections was minimal because the strains in the pipes and longitudinal bars were well below the yield, and cracks in the column and pedestal were thin and few.
4- In the cases that strands were used instead of the rod for the tension member, no yielding was observed in the strands. While the axial force was well below the yield force in the rod, the Von Mises stress passed the yield criteria due to the combination of flexure and tension.
5- The moment-rotation relationship of rebar-pins was stable even when the pins underwent large plastic deformations under many cycles of earthquake loading. The concrete near the hinge throat was damaged but the column and pedestal reinforcement did not yield near the rebar-pins.
6- Softening behavior was observed in the rebar-pins with axial load level more than 20% of the maximum axial force capacity of the hinge.
7- The reduction of the rebar-pin core diameter increased the displacement ductility of the bent and reduced the pin moment. However, stiffness and strength of the hinge degraded when the core diameters was reduced below 50% of the section diameter.
8- Similar performance was observed in the cast-in-place and the precast columns, which were built using a precast shell and filled with self-consolidating concrete (SCC).
9- The pocket connection using corrugated steel pipe and longitudinal bars extended for approximately 1.2 times the column diameter performed well in forming the plastic hinge in the column. No damage was observed in the pocket connections.
10- The column plastic hinges with ECC showed significantly less damage compared to the counterpart plastic hinges with conventional concrete even when the ECC was used in the column shell.

8. Conclusions
The following conclusions were drawn from the results of the experimental and analytical studies presented in this document:

1- The design and detailing methods developed and used in this study for both the rebar-pins and pipe-pins as well as the pocket connections and the precast cap beams were effective in leading to a ductile bridge bent even under extreme earthquakes.
2- The two-way hinge moments must be taken into account to avoid column shear failure and damage to capacity-protected members such as pile-shafts.

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3- Pipe-pins can be designed to remain entirely elastic when a strand is used for the tension member and may be treated as capacity-protected connections, whereas rebar pins are expected to yield.

4- Tension members should be used in pipe-pins to maintain global stability of the bent under larger lateral displacements, even in cases that the dead load is sufficiently large to prevent uplift.

5- Because rebar-pins undergo large plastic deformations, they should be designed as ductile elements with ample confinement for the concrete and sufficient development length for the reinforcement in the hinge.

6- In rebar-pins, slippage occurs even under small shear once a horizontal crack is formed across the entire section. The friction capacity increase with pin rotation and yielding of the reinforcement. This behavior is dominant in the pins with small axial force.

7- The proposed detailing for pocket connections was efficient and safe. In this detailing, a pocket with 1.2 times the column diameter is formed using a corrugated pipe, the spirals are provided around the corrugated pipe over the lower one-third of the pocket height, the column reinforcement is extended into the pocket, and the pocket is filled with self-consolidated concrete (SCC).