Seismic Design of Pipe-Pin Connections in Concrete Bridges

A dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy in Civil Engineering

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Abstract

Telescopic pipe-pin two-way hinges are used in concrete bridges to eliminate moments while transferring shear and axial loads from integral bridge bent caps to reinforced concrete columns. The hinges consist of a steel pipe that is anchored in column with a protruded segment that extends into the bent cap. In the absence of experimental and analytical studies, design of pipe-pin hinges has been based on pure shear capacity of the steel pipe. The primary objective of this research was two folds: (1) to investigate the seismic performance of the current detail of pipe-pin hinges and propose necessary modifications and (2) to develop a reliable design method for pipe-pin hinges that reflects their actual behavior. This research was comprised of comprehensive experimental and analytical studies of pipe-pin connections and their components including a shake table study of a two-column pier model.

The experimental component of the study included three sets of test models: (1) six push-off specimens to evaluate the bearing strength of concrete against the steel pipe, (2) six pure shear specimens to determine the yielding and ultimate shear capacities, and (3) a two-column 0.2-scale bridge pier model incorporating pipe-pin hinges that were designed based on the proposed guideline. The pier model was used to evaluate the new design method under earthquake excitation. The experiments showed that the lateral failure mechanism is typically controlled by concrete diagonal tensile cracking of the column in combination with flexural yielding of the steel pipe as opposed to pure shear, although the pure shear failure mode should be considered when a large amount of lateral steel is used in the column. Another possible mode of failure is bearing failure of the concrete around the pipe in heavily reinforced columns. The shake table experiment of the pier model confirmed that the proposed design method meets the safety and performance requirements under seismic loading.

The analytical studies consisted of (1) a stick model in SAP2000 that was developed for pipe shear key subassemblies, (2) detailed nonlinear FE models using ABAQUS that
were used to performed an extensive parametric study in order to shed light on different aspects of the behavior and generate the required data for the design guideline, and (3) a model in OpenSees that utilized a macro model for the pipe-pin hinges.

The experimental and analytical results helped identify the means to improve the performance of current pipe-pin hinge details. The pipe studs and spiral around the can proved to be unnecessary and were eliminated in the proposed standard detail. A thicker tapered hinge throat was suggested to solve the problem of local concrete damage to the throat and column edges. As a possible extension of pipe-pin application, a study was conducted on pipe-pins combined with isolation and damping systems. The analytical modeling of these details showed that modified connections can reduce the demands on the structure by dissipating a major portion of the earthquake energy.
Dedication

This dissertation is dedicated to my beloved mother for her unconditional support and love during all the stages of my life especially in my educational journey.
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1. INTRODUCTION

1.1. Introduction

Flexural hinges of various types have been used in reinforced concrete bridges since the beginning of the last century. The present trend towards using structural hinges in construction of new concrete bridges necessitates the advent of new cost effective, practical and reliable details. This task becomes even more challenging for concrete structures in seismically active regions.

Flexural hinges are intended to eliminate moment transfer between the connected members, while allowing for transfer of shear and axial force. One of the earliest applications of hinges was in arch bridges. In modern concrete bridges, the flexural hinges are mostly incorporated in either ends of the columns. A properly designed hinge, besides being sufficiently flexible to allow for rotation, should be capable of withstanding compressive forces as well as shear, be economical to construct, and require minimum maintenance [Kluge, 1940].

Articulating the columns in concrete bridges has distinct advantages particularly on seismic performance of the structure, including: 1) reducing the capacity demand on the cap beam or foundation and problematic soils [Cheng, and Saiidi, 2006], 2) reducing lateral stiffness or increasing the period of the structure; thus, reducing the seismic and the thermal forces, 3) reducing plastic shear in the column by converting double
curvature bending to single curvature one, 4) limiting joint shear in the panel zone, 5) help balancing stiffness when height of the columns are different (by hinging the short columns), 6) eliminating torsion in outrigger beams, 7) limiting damage to only one of the ends of column to facilitate post-earthquake repair, 8) simplifying seismic design of the bridge, and 9) reducing reinforcing bar congestion at the connection.

1.2. History of Concrete Hinges

Many different hinge details have been proposed for concrete bridges by researchers and engineers [Kluge, 1940]. These details work based on reducing the cross section of the concrete member, or by inserting a prefabricated part between the members, to reduce moment transfer from one member to another. Hinges usually incorporate steel sections, steel bars, or steel pipes to connect the parts. A brief review of some of the hinge details used in concrete bridges is presented in this section.

**Steel Hinges:** Early attempts to articulate concrete structures were mainly in arches and duplicated hinge details that were used in steel construction. Steel seats, attached rigidly to the concrete on each side of the hinge, were designed to rotate on a solid steel pin (Fig. 1-1) [Moreell, 1935]. This hinge only could release the moments about the pin axis; thus, it was a one-way hinge. Because of complications in anchoring these hinges to concrete and the need to protect them against corrosion, particularly when used at column bases, steel hinges can be expensive and impractical for concrete structures.

**Mesnager Hinge:** This detail can be considered as the origin of the current concrete hinges. This semi-flexible one-way or two-way hinge was first developed by an eminent
French engineer, Augustin Mesnager. In this detail, by crossing the steel bars at a notched section in the concrete member, the hinge is expected to effectively resist axial thrust and shear, while the flexural capacity and rigidity are significantly reduced (Fig. 1-2) [Moreell, 1935].

**Freyssinet Hinge:** Designed by Eugene Freyssinet; a French engineer, these hinges permit large rotations. This type of hinge is used in viaducts with inclined trestles and short columns supporting multi-span bridges. The hinge is made by forming a curved notch in the concrete member to reduce the moment capacity of the section (Fig. 1-3) [The highway agency, 1975]. Minimal or no reinforcement passes through the throat of Freyssinet hinges. It is recognized that the strength of the concrete in this hinge is considerable due to the confining effect of the wider adjacent member. However, the integrity of this hinge under high shear is of concern.

**Hinges with Reinforcing Bars:** As shown in Figs. 4a and 4b, these details result in a smaller effective cross section. These hinges are commonly incorporated in the column-foundation or column-bent cap connections. To release moment, the hinge is built in two forms. One is by stopping all the column longitudinal bars near the hinge and providing a smaller amount of separate longitudinal bars at the hinge throat, and the second is to cluster all or some of the longitudinal bars near the center of the section and pass them through the throat as straight or inclined bars, similar to those in Fig. 1-2. These details are intended to act as two-way hinges. Recent experimental and analytical studies have shown that, with proper design, two-way bar hinges could provide satisfactory performance under seismic loading [Saiidi, et al., 2009]. These studies have also shown
that, although the moment at the hinge is not zero, it is substantially lower than the moment capacity of the full section, and that the hinge moment needs to be accounted for in design.

1.3. Pipe-Pin Two-Way Hinges

Telescopic pipe-pin hinges were recently devised by bridge designers at the California Department of Transportation (Caltrans) to act as two-way hinges at the top of columns. These details are developed to completely eliminate moment transfer at the top of the column by preventing the development of any tensile stresses at the hinge section (Fig. 1-5).

Pipe-pin hinges are intended to transfer only shear and axial force across the column-deck joint. A concrete filled steel pipe that extends into an oversize steel can serves as the shear pin. The can is embedded into an integral cap beam (Fig. 1-6). The all-thread rod in Fig.1-6 is installed in the center of the pipe to prevent the steel can from floating during the casting of concrete in the pier cap. The rod has a small cross sectional area and does not provide any significant tensile force transfer across the hinge. A gap between the steel pipe and the can enables the protruded pipe rotate inside the can. A rubber gasket prevents the concrete from flowing into this gap. The gravity loads from the superstructure are transferred to the column through an elevated circular concrete bearing area around the steel pipe (hinge throat). As a result, the steel pipe is not subjected to any axial forces. A major portion of the lateral force is transferred via mechanical engagement of the pipe and the can. The friction on the hinge throat also
contributes to the lateral load transfer, although the magnitude of the friction force may decrease over time due to possible cyclic thermal movement.

1.3.1. **Pipe-pin Standard Details**

Caltrans has developed two standard pipe-pin details: Design-A and Design-B with different pipe sizes and pipe embedment lengths in the column (Fig. 1-7). Design-A is the smaller of two and comprises a 14-in (356-mm) diameter, 1.25-in (32-mm) thick steel pipe, which is embedded 60 in (1524 mm) into the concrete column. This detail has been used in columns with diameters ranging from 60 in to 84 in (1524 mm to 2134 mm). Design-B is a 20-in (508-mm) pipe with a thickness of 1.5 in (38.1 mm) and 84-in (2133-mm) embedment and is incorporated in columns with diameters larger than 84 in (2134 mm). In both details, a 0.5-in (12.7-mm) side gap is provided between the pipe and the can to allow the pipe rotate.

Two separate confining spirals are provided around the pipe and the steel can (Fig. 1-7). A series of headed studs are welded to the steel pipe to improve bond between the pipe and concrete in the column. When large rotations are expected at top of the column (i.e. tall columns), a larger diameter hinge throat (concrete bearing area) is provided to accommodate large rotations and prevent contact between the column edge and the soffit. The thickness of the hinge throat is limited to 1 in (25 mm) to minimize the risk of damage to the edge of the throat because the hinge throat is inaccessible for repair after earthquakes.

1.4. **Review of Related Research**
Extensive research has been carried out on bar hinges at UNR in the past [Saiidi, 1988, 1995, 2009]. Previous studies at UNR demonstrated that two-way bar hinges do not fail in pure shear contrary to what is assumed in their design, which is based on the shear friction theory. The large rotations at the hinge cause tension in the majority of the bars and compression in others [Saiidi, 1988, 1995, 2009].

There are some other instances in which steel pipes were incorporated to transfer shear between concrete members as shear lugs. Frosch [1999] used steel pipes to connect concrete shear panels to retrofit existing concrete frames. Four specimens were tested in his study to investigate the connection between precast infill wall panels and elements of the existing building frame. The study demonstrated that the lateral capacity could be determined by the shear yield strength of the pipe. Localized bearing failure occurred in the wall adjacent to the pipe at the interface because bearing stresses were not uniform over the depth of the pipe. It was concluded that the local failure did not affect the capacity of the section but did increase slip level required to regain the shear capacity.

Steel pipe shear keys were also used in the San Francisco Bay Area Rapid Transit (BART) aerial guideway structures to connect the girders to the closure pour (horizontal pin) and the closure pour to pier caps (vertical pins). Restrepo [2005] conducted a set of experiments on four pure shear specimens to proof test the lateral capacity of the vertical and horizontal pins. The general mode of failure in the specimens was local bearing failure of concrete next to the steel pins and the exterior cans with shear or flexural deformation of the pin itself. An empirical equation was proposed for nominal shear capacity of steel pipe shear keys accounting for effect of concrete inside steel tubes.
Dowel bars were realized as the closest similar structural components to pipe-pins when they are subjected to shear loading. Several researchers have investigated different aspects of the dowel effect. In dowel bars, ultimate transverse capacity is associated with failure of the surrounding concrete in bearing and flexural yielding of the bar [Soroushian, et. al, 1986, 1987, 1988; Vintzeleou, 1986, 1987, 1991; Marcus, 1951; Mannav, et. al, 1999; Dei Poli, 1992].

Design codes for concrete structures address the failure of anchor bolts or dowels bars when they are located close to the edge of concrete body [ACI 318, 2009]. In this case, formulation of lateral capacity of the anchor bolts is based on fracture of concrete in the form of a conical wedge. It is critical to realize that in dowel bars and anchor bolts, lateral mode of failure (bearing failure concrete of fracture of concrete) depends on distance between the dowel bar and the free edge of concrete body in direction of loading [Soroushian, 1986]. It was realized that for the pipe-pins, concrete failure would be the controlling factor for the lateral capacity; however, no relevant experimental data were available.

The lateral bearing of pipes against concrete is similar to the mechanism in extended pile shafts when subjected to lateral loads [Priestley, 2000; Chai, 2002]. It is well understood that lateral failure of the pile shafts is associated with the failure of the soil next to the pile under passive pressure of the pile, which coincides with flexural plastic hinging of the embedded pile at a certain depth.
As far as pure shear capacity of the pipe-pins is concerned, concrete-filled-tube (CFT) columns can be assumed as similar cases. Some researchers have studied the shear capacity of CFT columns in the panel zone (beam-column connection area) [Fukumoto, 2005; Fujimoto, 2004]. Yet, the geometry of the panel zone in CFT columns is different from pipe-pins; therefore, the equations proposed for shear capacity of the panel area would not be applicable to pipe-pins. This was the motive to do a series of pure shear tests on the infilled pipe specimens and develop a specific empirical equation in this research.

1.5. **Pipe-Pin Two-Way Hinges Versus Pipe Shear Keys**

Pipe-pins are mainly used as two-way hinges in concrete columns. This detail is also a potential alternative for conventional concrete shear keys. Caltrans uses steel pipe shear keys, referred to as “pipe seat extenders”, for retrofitting existing bridges. Pipe seat extenders are used on short seat hinges (<6 in, 152.4 mm) in order to prevent unseating [Caltrans, 2008; Roberts, 2005]. The seat extender are typically 8-in (203.2 mm) diameter xx-Strong un-grouted pipes, fixed at one end and free to slide at the other end. The fixed end of the pipe is usually the bearing side of the existing hinge [Hipely, 1997]. Pipe shear keys are also used by BART in aerial guideway structures as was explained [Tseng, 1992].

There are two basic differences between pipe-pin two-way hinges (column hinges) and pipe shear keys: 1) in pipe-pin hinges, large relative rotations are expected at the hinge due to the flexural deformation of the column, but in shear keys, lateral sliding is the only
possible movement, 2) pipe-pin hinges are located at top of the columns; thus, surrounded by a limited volume of concrete. On the other hand, shear keys are usually embedded in a massive body of concrete in superstructure or abutment.

The main focus of this study was on pipe-pin hinges, but the acquired experimental and analytical data were also applicable to pipe shear keys. To prevent any confusion, “pipe-pin hinge” and “pipe shear key” will be used to refer to column hinges and other shear keys, respectively.

1.6. **Objectives and Scope of Work**

The current design codes do not include any provisions for design of pipe-pin hinges. The method that is currently used to design pipe-pins is intuitive and only based on the pure shear capacity of the section of steel pipe. This study was aimed at a comprehensive understanding of the behavior of pipe-pin connections under seismic loading and developing practical design guidelines that realistically represent the actual behavior and failure modes of these connections. The study consisted of testing different types of test models and an extensive analytical investigation of various parameters.

1.6.1. **Experimental Studies**

To understand the lateral load transfer mechanism of the pipe-pin, a series of experiments was designed and performed at the Large Scale Structure Laboratory at the University of Nevada, Reno. The experiments were aimed at investigating the validity of a hypothesis that pipe-pin failure is associated with bending of the pipe and not merely pure shear failure of the pipe. These experiments included pseudo dynamic testing of a single
column model incorporating a pipe-pin hinge at the top to understand the general
performance of pipe-pin connections [Doyle, 2008], six push-off specimens to measure
the bearing strength of concrete against pipes, six pure shear concrete filled pipe
specimens to formulate the shear capacity of infilled steel pipes, and a two-column pier
model utilizing pipe-pins at the top of the columns to evaluate the validity of the
proposed design pipe-pin hinge design method.

The experiment on the single column model (PF-1) was carried out in a previous study
Doyle and Saiidi [2008]. Some of the conclusions and results from that work are used in
this report to help verify the analytical models.

1.6.2. Analytical Studies

In addition to the experimental investigations, an extensive analytical study of different
models was performed in this study. The analytical phase of the study could be divided
into three parts, each serving a different purpose.

First, a simple nonlinear stick model was developed in SAP2000 [Computer and
Structures, Inc., 2007] comprising lumped plastic hinges to study different aspects of pipe
shear keys. The results of this model were compared against the results of the push-off
experiments. Next, an elaborate nonlinear finite element (FE) model was constructed
using ABAQUS [Hibbit, Karlsson, and Sorensen, Inc., 2007] and calibrated versus the
experimental results of push-off and PF-1. This model was used to realize the effect of
many parameters that could potentially influence the performance and capacity of pipe-
pin hinges. The results generated by FEM analysis was used to develop an iterative
design guideline for the pipe-pin hinges. Finally, an OpenSees [Mazzoni, et. al, 2007] model was utilized to design the two-column bent specimen and develop the seismic loading protocol.

1.7. Document Layout

This chapter is followed by explaining the details of the experimental studies, comprising specimen drawings, test set up details, material properties, instrumentation, and loading protocols. Chapter two provides details of all the test models.

In Chapter 3, observations and the collected data from the experiments are presented. This chapter includes the general observations, load-displacement responses, strain data, and other recordings from the instruments. The empirical formula for the concrete bearing strength and pure shear capacity of infilled pipes are presented in this chapter.

Chapter 4 is allocated to the analytical studies. This chapter starts with introducing the stick model for shear keys. Then, the ABAQUS models are explained and at last the OpenSees model of the bent is presented. The comparison of the analytical data and experimental results are also presented in this chapter.

In Chapter 5, the results of the parametric study on the pipe-pin details are presented and explained in detail. The observed failure modes are also explained in this chapter.

Chapter 6 explains how the design method was developed for the pipe-pin hinges and the pipe shear keys. The theoretical concepts that support the design method are presented along with the design procedure.
In Chapter 7, several possible modified versions of pipe-pin hinges are presented that include an isolation system and damping devices. The issue of uplift in pipe-pin hinges is also addressed in this chapter.

The content of the document is summarized in Chapter 8. This chapter also presents a list of observations and important conclusions.

Five appendices, A, B, C, D, and E, are included in the document to present the design of the pipe-pin hinges in the bent specimen, strain gauge data from push-off specimen tests, strain data from the bent test, OpenSees code for the bent model, and a design example, respectively.
2. EXPERIMENTAL STUDIES

2.1. Introduction

Details of the experiments are explained in this chapter. This study included three experimental phases: push-off specimen tests, pure shear specimen tests, and a two-column bent proof test. For each phase the concept and purpose, details of the specimens, design considerations, material characteristics, instrumentation, test setup, and loading protocol are described.

2.2. Push-Off Specimens

2.2.1. Introduction

If the pipe-pin connection fails in concrete bearing, the resisting bearing stresses should be known in order to estimate the lateral load capacity of the connection (Fig. 2-1). The push-off specimens were constructed to quantify the bearing strength of concrete when pushed by the steel pipe-pin. It is well known that confinement increases the compressive strength and ductility of concrete. The same is true of local forces acting on a small area of a large body where the massive surrounding concrete provides confinement for the locally loaded area.

There are limited research studies that address bearing strength of concrete. The available experimental results are mostly restricted to the study of dowel action of embedded reinforcing bars. Dowel bars have much smaller diameter compared to the
pipe-pins (less than 1 in (25 mm) verses 12 to 20 in (300 to 500 mm)). Furthermore, the proposed values by different investigators vary substantially. For the bearing strength, different researchers have suggested a concrete bearing strength ranging from 1.45 to 5.0 times the concrete compressive strength [Soroushian, et al., 1988].

Concrete compressive strength, size of the loaded area, additional confinement provided by reinforcement, flexural strength and rigidity of the steel bars, and aggregate size are the recognized parameters that affect bearing strength of concrete. It is proven that the bearing strength of concrete increases by reducing the size of embedded object and increasing the concrete compressive strength [Dei Poli, 1992; Vintzeleou, 1990; Soroushian, et al., 1988; Marcus, 1951].

To obtain data on the bearing strength of concrete against steel pipes, six 1:3.5 scale push-off specimens were tested at UNR.

### 2.2.2. Test Specimens

Three different details were tested to formulate the bearing strength of concrete against the steel pipe-pin. To ensure that any scatter in the data is captured, one pair of specimens was built for each detail, thus six specimens were built. The specimens were labeled PS1P-A/B, PS2P-A/B, and PS3P-A/B. Figures 2-2 to 2-13 show the construction plans and details of the specimens. Table 2-1 summarizes the characteristics of the push-off specimens.

The test variables were the pipe diameter and the confinement around the pipe. Two different steel pipe diameters were used to study the effect of size. Specimens PS1P-A/B
and PS2P-A/B with dimensions of 18 x 36 x 48 in (457 x 914 x 1118 mm) incorporated 3-1/2 xx-Strong (“xx- Strong” stands for double-extra strong pipe) steel pipe with the outer diameter (OD) of 4.0 in (101.6 mm) and 0.636-in (16.15-mm) thickness. The small specimens; PS3P-A/B, with dimensions of 14 x 28 x 36 in (356 x 711 x 914 mm) included 2-1/2 xx-Strong with 2.88-in (73.1-mm) OD and 0.552-in (14-mm) thickness. In spite of the fact that the diameter to thickness ratio of prototype pipes is closer to that of the x-Strong steel pipes, xx-Strong pipe with a larger thickness was used in the tests to ensure that the pipe will not fail in shear.

An extra confining spiral is provided around the steel pipe in the Caltrans design. For PS1P-A/B and PS3P-A/B these inner spirals were W2.9@1 in (25.4 mm) and W1.7@0.75 (19 mm), respectively which represents #5@3 in (76.2 mm) in prototype. The inner spiral volumetric steel ratio for PS1P-A/B and PS3P-A/B was 1.37% and 1.45%, respectively, which was close to 1.38% in the prototype. In PS2P-A/B this spiral was removed to study its significance. Three galvanized steel wire sizes were used as spiral: W1.7, W2.9 and W5 with diameter of 0.148 in (3.76 mm), 0.192 in (4.9 mm), and 0.248 in (6.3 mm), respectively. PS1P-A/B and PS2P-A/B had an outer spiral of W5@1.5 in (38.1 mm). In PS3P-A/B the outer spiral was W2.9@1 in (25.4 mm). The outer spiral represents the column spiral in the prototype. The geometric ratios such as the ratio of spiral cage diameters to pipe diameter and the ratio of specimen width to pipe diameter were obtained by averaging and scaling the corresponding values of the pipe-pin details used in the San Francisco Bay Bridge approach spans.
Push-off specimens were symmetric with a gap in the middle (Figs. 2-2 and 2-8). The gap generated a constant eccentricity for the lateral load to account for the eccentricity of the resultant of the distributed contact pressure on the exterior can. This eccentricity is defined by $e$ in Fig. 2-1. The gap was formed by placing a Z-shape 3-in (76.2-mm) and 2.5-in (63.5-mm) thick Styrofoam sheet for PS1&2P-A/B, and PS3P-A/B, respectively (Fig. 2-14). The sheet was removed before the test. Symmetric design of push-off specimens eliminated the undesired local effects of the loading, because each half played the role of loading system for the other half.

2.2.3. Specimen Design

It was critical that the capacity of the specimens do not exceed the maximum loading capacity of the loading frame. To estimate the capacities, a preliminary finite element analysis was performed. The push capacity of the PS1P and PS2P was estimated to be approximately 110 kip (758.4 kN) which was less than 220 kip (1517 kN), the machine capacity.

The push-off specimens were initially designed to be tested only in the push direction; therefore, only two 0.75-in (19-mm) diameter all threaded anchor bolts were provided at each end to connect the grip plates to the specimen (Figs. 2-2, and 2-8). But, later the pre-analysis results indicated that the behavior would be significantly different in the push and pull directions and it was decided to also test the specimens in pull. The existing anchor bolts were unable to carry more than 30-kip (133.4-kN) tension; therefore, an extra external fastener was added to the specimens that allowed for a larger
pull force. The external fastening system was composed of a steel angle passing through the middle gap, and two high strength threaded rods. The details of the external fastener are illustrated in Fig. 2-13. The total tensile capacity with additional anchoring system was approximately 70 kips (311.4 kN) that dictated the maximum pull force.

Another issue that was considered in push-off specimens was out of plane stability. The only connecting element between the two concrete segments of the push-off specimens was the steel pipe with relatively small torsional stiffness. To ensure that the specimen does not deform out of plane under push forces, a set of steel plates were added to the sides of the specimens to restrain any movement except in plane relative sliding, as shown in Fig 2-2 as “Detail-A”. Figure 2-15 shows one of the T-shape steel parts with slotted holes on one side to allow for movement.

2.2.4. Material and Construction

After fabrication of the wooden forms, the Z-shape Styrofoam was fixed in place by gluing it to platform. Then the bar cages were inserted into the form and finally the steel pipes that were filled with concrete in advanced were slid into the form through a hole that was provided on one of the sides. Figures 2-16a, b show the bar cages in PS1P and PS2P, respectively. The final status of the specimens before casting concrete is shown in Figs. 2-17 to 2-19. The concrete in all the specimens was placed from the same batch outside the structural lab on the construction platform (Fig. 2-20, and 2-21). Figure 2-22 shows how the steel pipes were filled with concrete before being placed in the form.
The compressive strength of concrete for the specimens (except for the pipe infill concrete) at 7-day, 28-day, and day of the first test (103 days after pour) was 4.26 ksi (29.38 MPa), 5.51 ksi (37.96 MPa), and 5.91 ksi (40.77 MPa), respectively. The split cylinder tensile test results (Fig. 2-23) were 0.432 ksi (2.98 MPa) and 0.476 ksi (3.28 MPa) at 28 days and day of the first test, respectively. The concrete inside the pipe was only tested on day of the first test, and the compressive strength was 3.82 ksi (26.35 MPa).

The mechanical properties of steel materials are presented in Table 2-2. Figures 2-24 to 2-26 show the full stress strain behavior of the steel wires used as spirals. The wires were tested at material test lab at UNR, but the material properties of the steel pipes are those in the material test report (MTR) provided by the supplier.

2.2.5. Instrumentation

Figure 2-27 shows the configuration of the strain gauges installed on the steel pipe and concrete surface. The strain gauges were installed on both faces of the steel pipe. Only the pipe in one of the concrete segments was instrumented due to symmetry. They were intended to determine the strain profile along the length of the pipe and to locate any flexural yielding. A rosette strain gauge was installed on the pipe at the hinge throat (in the middle of the gap) to measure shear strains. Type of the pipe strain gauges and rosette was YFLA-5-3L and YEFRA-5-3L, respectively. For concrete surface a longer strain gauge (PL-60-11-3L) was selected to measure the average strain in a finite length instead of local strain that could be affected by concrete cracking. Strain gauges were
also attached to the spirals to investigate their role in carrying the load (Fig. 2-28).
Due to the symmetry of the specimens, only one side of each specimen was instrumented.

Another important data to monitor was relative displacement of two segments. The two
sides of the specimen were free to slip, move apart, and rotate relative to each other. A
shear panel combination of five Novotechnik linear displacement transducers was used to
capture these relative movements. Several other Novotechnik transducers were attached
to the specimen to measure any out of plane relative displacement. Figure 2-29 shows
the position and configuration of the Novotechnik transducers. Figure 2-30 shows one of
the shear panels.

2.2.6. Test Setup and Loading Protocol

The push-off specimens were tested using a MTS universal load frame at the UNR Large
Scale Structure Lab. This machine is able to apply up to 220 kips (980 kN) tensile and
compressive forces. The specimen was fixed in the frame by gripping the steel plates
attached to each end of specimen. Figure 2-31 shows how the specimens were placed in
the loading frame.

Each specimen was first pulled. In PS1P-A, B and PS3P-A, B, the head anchor bolts
started to pull off before reaching to yield force of the specimen and their capacity
controlled the maximum pulling force. For PS2P-A/B pulling continued until the load-
displacement curve showed large post-yield displacements.
After the pull, the load was returned to zero. Then, the compressive force was applied and continued far beyond yielding. For most of the specimens, the test continued until the gap between two sides closed as pointed by an arrow in Fig. 2-32.

2.3. Pure Shear Specimens

2.3.1. Introduction

Shear failure of the pipe is one of the possible failure modes under lateral loads. Researchers have already studied the shear failure of the bare steel pipes [Choo, et. at, 2002]. However, in pipe-pins, the concrete inside the steel pipe also participates in shear capacity; therefore, using the design formula proposed for steel pipes could underestimate the shear capacity of the in-filled pipe. A series of experiments were designed to investigate the effect of concrete inside the pipe on the pure shear capacity.

Most of the available technical literatures that address the shear capacity of the concrete filled tubes (CFT) focus on beam-column moment connection zones [Fukumoto, 2005]. However, depending on the size of the connecting beam to the column, the ratio of the shear span to diameter of the pipe is much larger than of the pipe-pin connections. As part of this research, the pure shear capacity of the concrete-filled steel pipes was experimentally investigated, and the results were summarized into simple design equations.
2.3.2. **Test Specimens**

Three pairs of 12-in (305-mm) long concrete-filled steel pipes were tested in double shear to measure the pure shear capacity of infilled steel pipes. These specimens were labeled IPS-1A/B, IPS-2A/B, and IPS-3A/B. The test variables were the diameter and thickness of the steel pipe. IPS-1A/B employed a 3-1/2 Standard steel pipe with 4-in (102-mm) O.D. and 0.226-in (5.7-mm) thickness. The corresponding values for IPS-2 were 4 in (102 mm) and 0.318 in (8.1 mm). IPS-3, the smallest specimen, was made up of a 2.88-in (73-mm) diameter pipe with 0.276-in (5.16-mm) thickness. Table 4 presents the geometric details and specifications of the shear specimens. Figures 2-33, 34, and 35 show the details of the pure shear specimens, and Fig. 36 shows the steel pipes before casting concrete.

2.3.3. **Material and Fabrication**

The compressive strength of concrete was 4.0 ksi (27.6 MPa) and 5.1 ksi (35.2 MPa) on 28 days and test date, respectively. The mechanical properties of steel were based on MTRs and are presented in Table 2-3.

2.3.4. **Instrumentation**

Only was one rosette strain gauge attached to the mid shear span in each pair to measure the extent of shear yielding. One Novotechnik transducers was installed on each side between the top saddle and bottom rigid sole plate to measure deformations. The average of the readings from two Novotechniks was used as shear deformation.
2.3.5. Test Setup and Loading

To apply the shear force, a set of three fitted half-circle high strength steel (Gr. 1018) saddle was utilized. Figure 2-37 shows the saddles and Fig. 2-38 shows the loading setup. Two 0.25-in (6.35-mm) steel pencil rods were welded to the sole plate to restrain the bottom saddles from moving during the test.

Analytical models demonstrated that effect of shear span on the behavior was significant; therefore, it was critical to use a shear span that represents those in a real pipe-pin detail. Based on the preliminary FE analysis of the pipe-pin details a clear shear span of \(0.5D_p\) was selected.

The 220-kip MTS loading frame was used to test the infilled pipe specimens in shear. Load was increased gradually well beyond the yield point. When testing the largest pipe thickness (IPS-2), the load reached to the maximum capacity of the machine and the testing stopped at that point.

2.4. Two-Column Bent Specimen

2.4.1. Introduction

As is discussed in Chapter 6, a design method was developed for pipe-pin hinges as part of this study. The design method was proof tested in a two-column bridge pier model. The experiment was designed to evaluate the performance and safety of a pipe-pin hinge designed by the proposed method. This large scale structural model was subjected to
simulated seismic ground motions on one of the three biaxial shake tables of the UNR Large Scale Structure Lab.

2.4.2. Test Specimen

A scaled two-column bridge pier was constructed for the last part of the experimental studies. This specimen was labeled PPTC, which stands for pipe-pin two-circular-column. Structural drawings of the PPTC specimen are presented in Figs. 2-39 to 2-45. The reasons for using a two-column bent versus single column specimens and details of the specimen are explained in subsequent sections.

2.4.2.1. Two-Column Bent vs. Single Column Specimen

The pipe-pin proof test specimen was a two-column pier. For a hinged column, two-column bent offered several advantages over single column specimens. First, a bent model did not require additional lateral restraining structure. Pipe-pin detail is torsionally unstable after the friction between concrete surfaces releases. For a single column specimen incorporating pipe-pin hinge, a complicate lateral supporting frame must be provided to restrain the out of plane rotation of the column head. However, in a two-column bent this problem did not exist.

Another advantage of a two-column bent in comparison with a single column was using only one link between the mass rig and the specimen (Fig. 2-39). In a real multi-column bridge, the bridge deck does not significantly rock about longitudinal axis of the superstructure and only sways laterally in an approximately horizontal plane. In the test model, the loading head represents the bridge superstructure and should remain
essentially horizontal when moves laterally. To duplicate this effect in a single column specimen, two parallel links are required to restrain the column head rotation. However, the bent cap in a two-column bent remains essentially horizontal and a single link is sufficient.

Moreover, a two-column bent provided the possibility to put two different column details into test in a single experiment. To do that, it was necessary to separately measure the base shear of each column as will be explained in next section.

2.4.2.2. Column Types

The primary objective of testing of PPTC was to evaluate the proposed pipe-pin hinge design. The presence of two columns in the pier provided an opportunity to study two different columns; although, the pipe-pin design was the same for both columns. By placing a load cell in the middle of the cap beam, it became possible to measure shear in each column (Fig. 2-41, and 2-65). Because of the hinge action of the pipe-pin, no moment is transferred from the columns to the cap beam; thus, the moment in the beam is minimal and effect of the load cell in global response is negligible.

It was necessary to build one of the columns with conventional detail to study the performance of the pipe-pin hinges designed by the proposed method. The choice of the other column was based on the fact that pipe-pin hinges provide an opportunity for new structural systems. Thus, the other column was constructed using a new column system that was developed by Mirmiran [2003] in order to study the performance of this system in combination with pipe-pin detail. The new column was a concrete filled fiber
reinforced polymer (FRP) tube element. In this tube, glass fibers serve both as longitudinal and transverse reinforcement. To increase the flexural capacity and energy dissipation, a small amount of longitudinal reinforcement was used in the column. No lateral steel reinforcement was used because the entire shear and confinement requirements were provided by the FRP tube.

2.4.3. Model Scaling Factor

Size and reinforcement vary significantly in real bridge columns incorporating pipe-pin details, but columns tend to be of relatively large cross sections. Table 2-4 presents detailing information for sample column types that are used in the replacement of approach spans of the San Francisco Oakland Bay Bridge. The columns are square but the longitudinal bars are placed in a circular pattern. Therefore, they were treated as circular columns in this study. The column diameter ranges from 5 ft to 7 ft (1.524 m to 2.133 m) for a 14-in (355-mm) pipe-pin, but the columns incorporating the 20-in (508-mm) pipe-pin have 8-ft (2.44-m) diameter. By averaging, prototype was assumed to be a 70-in (1778-mm) diameter column with Design-A pipe-pin of Caltrans which has a 14-in (355.6-mm) O.D. and 1.25-in (31.75-mm) thickness. The proposed detailing recommendations of Chapter 6 were used to determine other dimensions such as pipe length, gap thicknesses, and etc. The column length to diameter ratio in the model was assumed 4.25 to generate relatively large shear demand on the pipe-pin while allowing for considerable rotation at the top of the columns.
The geometric scale for the shake table model of the prototype was 0.20. The model for shake table testing needed to be as large as possible and at the same time be able to reach failure without exceeding the capacity of the University of Nevada, Reno shake tables. Using realistic materials in the model would provide a better match to the actual prototype response. One-fifth scale allowed the pipe-pin to be sufficiently large to accommodate regular concrete instead of micro-concrete.

To ensure that the shake table was of sufficient capacity to fail the bent model, the demands were compared to capacities of the shake table using a computer code specifically developed for the UNR shake tables. Specifications of the UNR shake tables are listed in Table 2-5.

2.4.4. Specimen Design

2.4.4.1. Design Concept

Design codes allow for only ductile modes of failure. As an example shear capacity of the column must be always larger than maximum plastic shear (the lateral load associated to the maximum flexural capacity). The same concept was observed for pipe-pin connections. Pipe-pin hinges are designed to remains elastic while the column undergoes plastic deformation. No yielding or energy dissipation is expected to occur at pipe-pin hinges, and they are only supposed to act as force controlled members that transfer the lateral inertial force of the deck to the columns. Similar to other connections, pipe-pin hinges are intended to be capacity protected.
The main reason for preventing the pipe-pin to yield is that access to the hinge is not possible to repair the damaged pipe-pin after earthquake, unless the superstructure is completely lifted. For the two-column bent model, the pipe-pin was designed to be stronger than the plastic shear of the columns associated with plastic moments that were amplified by overstrength factor of 1.2, as defined by Caltrans seismic design criteria for shear.

\[ \phi H_n \geq V_o \]  
\[ V_o = M_o/l_c \]  
\[ M_o = 1.2 M_p \]

Where

- \( \phi \) = Capacity reduction factor of the pipe-pin as will be explained in Chapter 6
- \( H_n \) = Nominal capacity of the pipe-pin
- \( l_c \) = Length of cantilever column
- \( M_p \) = Plastic moment capacity of the column

2.4.4.2. Flexural Design

Both columns had the same length of 59.5 in (1511.3 mm). The conventional concrete column had 14 in (355.6 mm) diameter with longitudinal steel ratio of 2.6%. A moment-curvature analyses showed that the plastic moment capacity of this column is approximately 38 kips (169 kN).
For the FRP tube, a 14.567 in (370 mm) diameter Red Thread® II pipe with wall thickness of 0.269 in (6.83 mm) was chosen from NOV Fiber Glass Systems company production. Red Thread II pipe is a filament wound product using epoxy resins and continuous glass filament with a resin rich interior surface that is used as piping in chemical plants. Fibers in this product are aligned in ±55°, which provide strength in the longitudinal and hoop directions. An available model for the longitudinal behavior of the FRP column was used in moment-curvature analysis [Mirmiran, 2003]. In this model a curvilinear behavior is assumed with a specific hysteretic behavior that will be explained in detail in Ch. 4. The longitudinal bar steel ratio in FRP column was 1.04%. This steel ratio led to comparable shear in both columns of the bent at approximately 5% drift. The estimated capacity of the FRP column at 12% drift was 48 kip (213 kN). Figure 2-44 shows the reinforcing details of the columns.

The axial load index (ALI) in concrete bridge columns range from 0.05 to 0.1. ALI is defined as ratio of the axial load to the product of the specified compressive strength of concrete and gross cross section area of the column. The column axial load was 50 kip (178 kN), which results to ALI=0.065 for the 14 in (355.6 mm) diameter column with specified concrete strength of 5.0 ksi (34.5 MPa). The total effective weight of the mass rig was 100 kip (444.8 kN) and corresponded to the total axial load of the columns.
2.4.4.3. Shear Design

Caltrans design guidelines [Caltrans, 2006] were followed for shear design of the conventional concrete column. The design shear capacity, $V_u$, was determined using the following equations:

$$\phi V_n \geq V_o$$  \hspace{1cm} (2-4)

$$V_n = V_c + V_s$$  \hspace{1cm} (2-5)

$$V_c = 0.8v_c A_g$$  \hspace{1cm} (2-6)

$A_g =$ Gross section area  \hspace{0.5cm} $\text{in}^2$ (mm$^2$)

$v_c =$ Concrete shear capacity by taking into account member target ductility and axial load  \hspace{0.5cm} psi (MPa)

$$v_c = \begin{cases} 
\text{Factor 1} \times \text{Factor 2} \times \sqrt{f_c'} \leq 4 \sqrt{f_c'} & \text{psi} \\
\text{Factor 1} \times \text{Factor 2} \times \sqrt{f_c'} \leq 0.33 \sqrt{f_c'} & \text{MPa} 
\end{cases}$  \hspace{1cm} (2-7)

$$\text{Factor 1} = \begin{cases} 
0.3 \leq \frac{\rho_s f_{sh}}{0.15} + 3.67 - \mu_d < 3 & \text{ksi} \\
0.025 \leq \frac{\rho_s f_{sh}}{12.5} + 0.305 - 0.083 \mu_d < 0.25 & \text{MPa} 
\end{cases}$$  \hspace{1cm} (2-8)

$$\text{Factor 2} = \begin{cases} 
1 + \frac{P_c}{2000A_g} < 1.5 & \text{psi} \\
1 + \frac{P_c}{13.8A_g} < 1.5 & \text{MPa} 
\end{cases}$$  \hspace{1cm} (2-9)

$$V_s = \frac{\pi A_b f_{sh} D'}{2s}$$  \hspace{1cm} (2-10)

$$\rho_s = \frac{4A_b}{D'S} \text{ Transverse steel ratio}$$
\[ f_{yh} = \text{Yield strength of transverse steel} \quad \text{ksi (MPa)} \]

\[ \mu_d = \text{Displacement ductility} \]

\[ P_c = \text{Axial load on the column} \quad \text{Lb (N)} \]

\[ A_b = \text{Spiral leg area} \quad \text{in}^2 \text{ (mm}^2) \]

\[ D' = \text{Diameter of central cord of spiral} \quad \text{in (mm)} \]

\[ s = \text{Spiral pitch} \quad \text{in (mm)} \]

No steel transverse reinforcement was used in the FRP column. The shear strength of the FRP column was determined assuming that hoop strength of FRP tube plays the role of steel spiral. To do so, these terms were defined:

\[ V_n = V_c + V_{FRP} \quad (2-11) \]

\[ V_{FRP} = \frac{\pi t_{FRP} f_{FRP,h} D_{FRP}}{2} \quad (2-12) \]

\[ \rho_{FRP} = \frac{4t_{FRP}}{D_{FRP}} \quad (2-13) \]

\[ f_{FRP,h} = \text{Tensile strength FRP tube in hoop direction} \quad \text{ksi (MPa)} \]

\[ D_{FRP} = \text{Outside diameter of FRP tube} - t_{FRP} \quad \text{in (mm)} \]

\[ t_{FRP} = \text{Thickness of FRP tube} \quad \text{in (mm)} \]

The concrete shear capacity was found using Eq. 2-6 to 2-9. The same modification factors that are presented in Eqs. 2-8 and 2-9 to account for ductility demand and axial load were used in the FRP column by replacing steel volumetric ratio with FRP volumetric ratio. Figure 2-46 shows the calculated shear capacity of the column versus
top displacement of the column. The lateral shear capacity from the FRP layer was relatively high, hence that shear capacity was not affected by ductility demand. The column shear capacity was larger than the demand by a factor of 1.48 and 1.78 for the conventional concrete and the FRP column, respectively for the estimated ultimate drift of 10%.

2.4.4.4. Pipe-Pin Design

Detailed design calculations of the pipe-pin hinges are presented in Appendix A. The original design method was developed for pipe-pin hinges in conventional concrete columns, but was modified for FRP tube column by replacing the column spiral with FRP tube parameters in formulations.

Design required a 2-1/2 x-Strong steel pipe as the pipe-pin for both columns. Outer diameter and thickness were 2.88 in (73.15 mm) and 0.276 in (7 mm), respectively. Embedded length of the pipe-pin was 13 in (330 mm) and the protrusion length was 3.5 in (89 mm). The gap thickness between the pipe-pin and the steel can was 0.15 in (3.8 mm), and the horizontal gap (throat height) was 0.25 in (6.35 mm). Exterior can thickness was 0.15 in (3.8 mm). Six 3/16-in (4.76-mm), 1-in (25.4-mm) long steel studs were welded on top side of the can to improve connection with concrete (Fig. 2-58). The diameter of the bearing area in the model was 8.5 in (216 mm) as shown in Fig. 2-57.

Design of the inner spiral led to a coil of W2.5@0.75 in (19 mm) with wire diameter of 0.178 in (4.52 mm) in and cage diameter of 8.5 in (216 mm). In the conventional concrete column, the column spiral that was designed for the bottom plastic hinge was
found to be adequate to achieve the pipe-pin target capacity. In the FRP column, no column spiral existed and hoop resistance of FRP tube played that role. The details are shown in Fig. 2-42 and 2-44. The dimensions of the prototype and model pipe-pins are presented in Table 2-6.

Design capacities of the pipe-pin hinges were 42.8 kip (190.4 kN) and 70.5 kip (313.6 kN) for the conventional concrete column and FRP column, respectively. Plastic shear demands for the pipe-pin hinges of RC column and FRP tube column were of 38 kip (169 kN) and 48 kip (213.5 kN), respectively.

2.4.4.5. Footing Design

In conventional concrete column-footing connection, the column longitudinal bars were extended to the footing and were anchored with a 90° standard hook. In the FRP tube column, the tube had to be properly anchored into the footing to transfer the plastic moments. Based on past research done by Mirmiran, 2003, the FRP column was embedded in the footing 1.5 times the column diameter (21 in, 533.4 mm). A study that was done by Mander et al. [2002] provides some guidelines for required embedded length of the steel piles inside the concrete footings. To find the embedded length that guarantee full flexural continuity, it is suggested to limit the stresses due to lateral shear and bending as shown on Fig. 2-47, to the concrete compressive strength. Equation 2-14 is proposed in his research for embedded length by replacing the properties of the steel pile with the ones of FRP tube.
\[ M_u \leq \frac{f'_{d_{FRP}} L_e^2}{6.3} \]  \hspace{1cm} (2-14)

\[ M_u = \text{Ultimate moment of the column} \quad \text{kip.in (kN.mm)} \]

\[ L_e = \text{Embedded length of the FRP column} \quad \text{in (mm)} \]

This equation result in the required embedded length of 16 in (406.4 mm) which is smaller than the provided embedded length of 21 in (533.4 mm).

The footing was designed to essentially remain elastic. Footing size was 48 x 132 x 24 in (1219 x 3353 x 610 mm). Minimum reinforcing steel was adequate to resist the ultimate moment of the conventional concrete column. For the FRP column an additional amount of longitudinal reinforcement was provided to resist the splitting force of the embedded part of the FRP column. Figure 2-47 shows the moment and shear force transfer mechanism between the bottom and the footing. The tensile force at top layer of reinforcement is:

\[ T_s = V_o + \frac{M_u}{0.8L_e} \]  \hspace{1cm} (2-15)

The required steel area to resist this force without yielding is:

\[ A_{s,\text{top}} = \frac{T_s}{0.6 f_y} \]  \hspace{1cm} (2-16)

\[ V_o = \text{Plastic shear at the column base} \quad \text{kip (kN)} \]

Two of the footing bars passed through the FRP tube in each direction.
2.4.4.6. Bent-Cap Beam Design

The bent cap was reinforced by 3-#5 bars at top and 4-#5 at the bottom. An even number of bars was required at the bottom layer to allow for the placement of the can in the middle. Four high strength 1-in (25.4-mm) diameter all threaded rods used to post-tension the beam segments to the middle load cell and the link swivel head. The total prestressing force of four rods was approximately 200 kip (890 kN). This post-tensioning force guaranteed that the load cell would not separate from the concrete beam.

2.4.5. Construction

The two-column bent was cast on the construction platform outside the lab. The footing was first poured after the bar cages and FRP tube were positioned and fixed in place. Figures 2-48 to 2-51 show the footing construction stages.

Placing the steel pipe and inner spiral inside the column in their right positions and securing that to assure they do not move during concrete pour was a challenging task. To ensure that the clearance between the inner spiral and the steel pipe and also between the inner spiral and column cage would not change, a small cross shape chassis made out of 0.125 x 1 in (3.175 x 25.4 mm) steel strips was tack-welded to the inner spiral cage and steel pipe as Fig. 2-52 illustrates. This system then was tack-welded to the column cage after it was plumped. Figures 2-53 to 2-56 show the final configuration of the pipe-pin on top of each column.

The hinge throat was cast by using a 0.25-in (6.35-mm) thick dense Styrofoam sheet as filler around the hinge throat. The Styrofoam was glued to the concrete surface using
epoxy (Fig. 2-57). After this step, the steel cans were located on top of the pipe and were fixed in place with a 1/8 in (3.2 mm) threaded rod embedded in the concrete inside the pipe, as marked by an arrow in Fig. 2-59. Figure 2-58 shows details of the steel cans. To make sure that the gap between the pipe and the can is even around the pipe, four 0.15-in (3.8-mm) thick wooden spacers were placed between the can and the pipe. Can bottom was sealed by silicon glue to ensure that concrete does not leak into the gap.

After the steel can was secured in place, the load cell was inserted into the form, bar cages of the cap beam was placed, and PVC pipes were passed through the length as ducts for pre-stressing all-threaded rods as Fig. 2-60 demonstrates.

2.4.6. Material Properties

Testing of the materials for the two-column bent served two purposes. The first was to make sure that the concrete and reinforcement had properties that satisfied the design and reflected the properties of the prototype columns. The second was to be able to use the measured constitutive relationships in the post-test analytical modeling. Testing was conducted for all the concrete parts of the model and for spirals and longitudinal bars.

2.4.6.1. Concrete

The concrete had a 3/8-in (9.5-mm) maximum aggregate size and was designed for an unconfined compressive strength of 5.0 ksi (34.5 MPa). Concrete was ordered with a specified 28-day compressive strength of 5.0 ksi (34.5 MPa), expecting an actual strength ranging from 4.5 ksi (31.0 MPa) to 7 ksi (48.3 MPa), based on past experiences. The small aggregate size was required because of relatively small pipe size and small cover
thickness due to small scaling factor of 0.2. A small amount of super plasticizer was added to each patch on site to increase workability without reducing the strength. Table 2-7 lists the test data for the concrete from each batch. There were three different castings: footings, columns, and cap beam. Standard 6 x 12 in (150 x 300 mm) cylinder compressive strength were measured at 7-day, 28-day, and at the end of the shake table tests.

Strength of the column concrete was the most important because of its effect on bent stiffness and strength. The measured compressive strength of the concrete used in the columns at the end of shake table testing was 6.87 ksi (47.4 MPa). The concrete compressive strength for the footings and the cap beam was 8.15 ksi (56.2 MPa) and 8.07 ksi (55.64 MPa), respectively.

2.4.6.2. Steel

Tensile testing was conducted for the lateral spiral wire reinforcement, W2.5 and W5.5, and the longitudinal #4 bar reinforcement in the columns. Grade 60 bar was used throughout the bent model.

Figures 2-61 to 2-63 show the measured stress-strain curves for #4 bars and the wires. The yield stress for the wire reinforcement tests was determined using 0.2% offset method. The extensometer that measured the bar extension did not perform well at the beginning of the loading; therefore, the initial part of the data was removed from the graphs. The average measured yield stress of the #4 bars, W2.5, and W5.5 wire was 68.8
ksi (751.5 MPa), 89.3 ksi (615.7 MPa), and 109 ksi (751.5 MPa), respectively. The measured elastic modulus was approximately 29000 ksi (2e5 MPa).

Because the wires were stretched by fabricator to achieve to the desired diameter (cold working), the steel wires were high strength. As will be discussed in the next chapter the measured strains in these wires were smaller than 2400 microstrains, which corresponds to a stress of 70 ksi (482.6 MPa); therefore, the high strength of the wires did not affect performance of the model.

The steel pipe and can material properties were obtained based on the MTRs provided by the supplier. The yield strength of the steel was 52.2 ksi (360 MPa) and 52.9 ksi (367 MPa) for the pipe and the can, respectively. The properties of the steel materials are summarized in Table 2-8.

2.4.6.3. FRP

All the material properties for the FRP tube were based on the supplier technical information. The mechanical properties of the FRP tube are provided for temperatures 75° F (24° C) and 210 ° F (99° C) by the supplier. Table 2-9 summarize the mechanical properties of FRP tube, but only the lower temperature is of concern in structural engineering. Two important values from Table 2-9 are the hoop strength and beam bending strength that are 23 ksi (158.6 MPa) and 34 ksi (234 MPa), respectively. Using the ultimate stress and the module of elasticity in bending, results to the rupture strain of 26500 microstrains.
2.4.7. Instrumentation

To monitor various aspects of the behavior, 189 channels of data were collected during the experiment. A large number of strain gauges, Novotechnik displacement transducers, strain potentiometers, load cells, and accelerometers were installed on the specimen. Figures 2-63 through 2-65 show the details of the instrumentation.

Details of the strain gauges are presented in Figs. 2-63 and 2-64. Strain gauges were installed on four of the outermost northern and southern longitudinal bars in both columns. The strain was monitored at five levels to help determine the length of plastic hinging and the extent of yielding in longitudinal bars. Eight strain gauges were installed on four sides of the conventional RC column at two levels on the spirals. Another set of strain gauges were installed in the pipe-pin hinge area in both columns.

Ten strain gauges were installed on the southern and northern faces of each steel pipe at five levels to detect pipe plastic hinging. Two rosette strain gauges were also installed on the eastern and western sides of each pipe at the hinge throat to measure shear strains. To determine the distribution of stresses around the pipe-pins, three levels of the inner spirals were instrumented using two strain gauges at the front and side at each level. In the RC column the same number of the strain gauges was also installed on the column spiral, but for the FRP tube column, the hoop strain was measured on the surface of the pipe at two levels using four strain gauges. To determine the distribution of flexural, hoop, and shear stresses on the FRP tube, ten longitudinal strain gauges were glued to the FRP tube surface at five levels, two inside the footing and three at 1 in (25.4 mm), 7 in (178 mm), and 13 in (330 mm) from the footing. The shear strains were measured at two levels on
the FRP tube using four rosette strain gauges. Two gauges were also installed on each steel can. In addition four gauges were installed on the longitudinal bars in the bent cap beam.

To measure curvatures and bond-slip rotations, 10 Novotechnik sensors were attached to the bottom of each column at four levels (Fig. 2-65). Six Novotechniks were also installed at the top of each column, four to measure the relative slip between the bent cap and the column, and two to measure relative rotations. Two Novotechniks were also installed to detect if any movement occurs between the two segments of the cap beam in the middle load cell area.

Figure 2-65 shows the configuration of the strain potentiometers that were installed on the specimen. Two strain potentiometers were installed between the cap beam and a reference frame to measure the absolute in-plane displacements. Two more were installed between the bent cap and the western wall of the lab to measure any out of plane movement. In addition, two sensors were installed between the cap beam and the footing to measure vertical movements of the cap beam.

An accelerometer was installed on the cap beam to measure the acceleration histories.

The lateral forces were measured using two load cells. One was part of the link assembly and measured the total lateral force including the P-Delta effects (because the P-Defects are generated by the mass rig). The second was a six-DOF load cell that was placed between the two cap beam segments. All the components of the load were recorded in this load cell but only the axial force was of concern. Four load cells were installed on
top of the steel axial load beams to measure the axial loads. Total of eight high
strength rods were used to apply the axial load on the column (four on each). The load
cells were placed on every other axial load rods in a zigzag pattern.

2.4.8. Test Setup

Setup of the two-column pier model included post-tensioning the bent cap and link
swivel head, removing the scaffolding, attaching the vertical load steel beams, placing the
vertical load high strength threaded rods, grouting the footing on the shake table,
clamping the footing to the table, and attaching the steel link to the cap beam and mass
rig.

Because of the hinges at the top of the columns, there is no continuity between the cap
beam and columns; therefore, extra attention had to be paid when moving the specimen.
Figure 2-39 shows the setup plan and Figs. 2-66 and 2-67 show the two-column specimen
on the table.

2.4.9. Input Ground Motion and Loading Protocol

Several characteristics were examined to choose the ground motion. The ground motion
had to place sufficient demand on the bent to allow for failure. Yet the displacement,
velocity, and acceleration demand imposed on the shake able needed to be within the
shake table limits. A relatively symmetric response was desired to study the performance
of pipe-pin connections under full reversed cycles. Several real and artificial acceleration
histories were reviewed to determine the record that meets all the requirements. It was
understood that many design earthquake engineers prefer real records over artificial ones
because the latter might place unrealistic demand on the structure. Upon the review of different records, data from one of the stations obtained during the 1994 Northridge, California earthquake was selected for the input ground motion.

The earthquake record used in the experiment was the modified version of the motion measured at the Sylmar Converter station. The Sylmar Converter station is located at 34.3110 Latitude, -118.490 Longitude on a soil layer with \( V_{s30} \) of 824.14 ft/s (251.2 m/s) with 13.11 km epicentral distance. More information regarding the earthquake and the station is presented in Fig. 2-68. The 142-degree lateral component of the acceleration history was used in the test.

The accelerogram was modified in two steps to produce the shake table target input motion. The motion was first filtered using SeismoSignal ver. 3.3.0 software, using Butterworth bandpass 4\(^{th}\) order filter for the frequencies higher and lower than 25 Hz and 0.2 Hz, respectively. The reason for filtering the ground motion was to achieve a symmetric ground displacement and a somewhat symmetric structural response. The corresponding gain function is plotted in Fig. 2-69. The acceleration, velocity, and displacement of ground for the first 15 seconds after and before filtering being applied are shown in Figs. 2-70, 2-71, and 2-72, respectively. The response spectra of the motion with 5% damping and the estimated period of the intact and damaged prototype structure are presented in Fig. 2-73. The time of the record was compressed by a factor of \( \sqrt{0.2} = 0.447 \) to take the effect of scaling into account.
The bridge was subjected to a set of seven progressive excitations, from low amplitude to high amplitude. The acceleration scaling factors were 0.1, 0.4, 0.7, 1.0, 1.3, 1.6, and 1.9. The corresponding target PGA values were 0.091g, 0.364g, 0.637g, 0.91g, 1.183g, 1.44g, and 1.729g for runs 1 to 7, respectively.
3. EXPERIMENTAL RESULTS AND OBSERVATIONS

3.1. Introduction

The results and observations of the experiments that were discussed in the last chapter are presented in this chapter. The contents of this chapter are the observed behavior, measured data, analyses of the raw experimental data, and the empirical relationships that were developed based on the measured data.

The data for each group of test models are presented separately. For the push-off model and the two-column bent, the measured strain data plots are shown in Appendix B and C, respectively.

3.2. Push-off Specimens

3.2.1. General Observations

3.2.1.1. Behavior under Pull Loading

As stated earlier, the loading began by applying a pull force on the specimens until either the cracks appeared around anchor bolts (PS1P-A/B and PS3P-A/B), or extensive shear cracking occurred around the steel pipe (PS2P-A/B).
Under the pull loading, the pipe was pressed towards the free edge of specimen. The resistance was similar to that provided by a column in a pipe-pin column hinge in that the pipe was surrounded by a finite body of concrete. Typically, damage was initiated by two cracks in the concrete, starting from the edge of the pipe. In Fig. 3-1 this crack is highlighted by a dashed line for better visibility and is marked by an arrow. Figure 3-2 shows these cracks after they propagated to the sides of the specimen. In PS1P-A/B and PS2P-A/B the extent of cracking was less because of the strength provided by the inner spirals around the pipe. In PS2P-A/B the side cracks were larger. In these specimens two more cracks were formed in front of the pipe and propagated towards the face of the concrete block (marked by arrows in Fig. 3-3). The larger extent of cracking in the specimens that lacked an inner spiral confirmed the role of the lateral reinforcement in resisting forces. No concrete crushing was observed next to the pipe during pull.

3.2.1.2. Behavior under Push Loading

The behavior of the specimens was completely different under push loading. Under push loading, the pipe was pressed towards a relatively large body of concrete. In this case the pipe behaved essentially as a shear key. The resistance mechanism is also similar to that provided by the bent cap in a pipe-pin connection. The damage started with flaking of concrete next to the pipe as Fig. 3-4 illustrates. This was the start of bearing failure of the concrete. By increasing the load, the concrete next to the pipe continued to crush and the pipe itself started to bend. Under larger loads, the concrete next to the pipe failed in bearing and spalled off in a seashell form as highlighted in Fig. 3-5. The damage pattern was the same in the specimens with or without the inner spiral. Figure 3-6 shows the
deformed pipe between the segments after the test. Schematic failure mode of the specimens under push loading is presented in Fig. 3-7.

3.2.2. Load and Displacement Response

In order to measure the relative displacement of two segments, a shear panel composed of five Novotechnik transducers was installed on the specimens (Fig. 3-2). A geometric transformation matrix was used to convert the data to the relative displacement in the loading direction. Figure 3-8 shows the local and global degrees of freedom in the shear panel. Equation 3-1 shows the matrix relating the local and global DOFs and Eq. 3-2 shows the expanded form.

\[
\tilde{T} \cdot \tilde{U} = \tilde{u} \quad (3-1)
\]

\[
\begin{bmatrix}
0 & 0 & 0 & 1 & 0 \\
0 & 1 & 0 & 0 & 0 \\
\sqrt{2}/2 & \sqrt{2}/2 & 0 & 0 & 0 \\
1 & 0 & -1 & 0 & 0 \\
0 & 0 & 0 & 0 & 1
\end{bmatrix}
\begin{bmatrix}
U_1 \\
U_2 \\
U_3 \\
U_4 \\
U_5
\end{bmatrix}
=
\begin{bmatrix}
u_1 \\
u_2 \\
u_3 \\
u_4 \\
u_5
\end{bmatrix} \quad (3-2)
\]

By multiplying two sides of the Eq. 3-1 with the inverse of the transfer matrix, the global displacements were obtained based on measured displacements. Having the nodal displacements in global coordination, the relative displacement between the segments can be calculated using Eq. 3-3.

\[
u = \frac{U_3 + U_1 - U_5}{2} \quad (3-3)
\]
To find the actual load, one half of the weight of specimen was subtracted from the load registered in the testing machine under pull force, but was added to the measured load under push force.

The measured force-displacement curves for the push-off specimens are presented in Fig. 3-9, 3-10, and 3-11 for PS1P-A/B, PS2P-A/B, and PS3P-A/B, respectively. Note that the results for each pair of specimens are approximately the same indicating that there was no scatter in the load-displacement data. The maximum pull forces were quite different because the tests were stopped at the point that cracking was detected around the anchor bolts. The average ultimate push load for PS1P, PS2P, and PS3P was 131.3 kip (584 kN), 127.6 kip (567.5 kN), and 76.6 kip (323 kN), respectively. Comparison of the strength for PS1P and PS2P shows that the inner spiral increased the push capacity only by 3%. The maximum pull loads were 69.7 kip (310 kN), 66.5 kip (296 kN), and 23.53 (104.7 kN) in PS1P, PS2P, and PS3P, respectively.

It is clear from load-displacement curves that the capacity was much larger under push loading than it was under pull loading. The difference is due to the fact that the mechanisms to resist the pull and push loads are different. Under pull loading, tensile cracking of the concrete controlled the capacity, but in push, the bearing failure of concrete governed the behavior.

The load-displacement curves for PS2P, in which no inner spiral was used, started to soften at approximately 0.1 in (2.54 mm) due to extensive cracking; while no softening
and yielding was detected in PS1P. This observation confirms that the inner spirals helped resist the loads when cracking of the concrete is expected.

Under push loading, the load-displacement curves consisted of three distinct parts as shown on Fig. 3-12 for PS1P-B and PS2P-B. The behavior was linear elastic in part (1). The nonlinear behavior began at start of part (2), with the main source of nonlinearity being the shear yielding of the pipe. In part (3) concrete began to crush and the bearing capacity of the concrete against the pipe controlled the behavior. A minor drop in the load can be seen in PS2P-B as pointed by an arrow in Fig. 3-12, while in PS1P-B the load continued to increase. The reason for the difference is believed to be the inner spirals in PS1P-B. This spiral provided extra confinement for concrete and improved its strength and ductility.

3.2.3. Measured Strains

The measured strains for the push-off test models are plotted in Appendix B. The labeling of the strain gages is shown in Fig. 2-27 and 2-28. These data are grouped into: shear strains on the pipe, normal strains on the pipe, surface strains on surrounding concrete, and strain on the inner and outer spirals.

3.2.3.1. Shear Strain on the Pipe

A rosette strain gauge recorded the shear strains of the steel pipe on the middle of the gap. Figure 3-13 shows one of these strain gauges that were installed on side of the pipe. The composition of three strain gauges that makes up the rosette gauges is presented in
Fig. 3-14. Using the numbering shown in this figure, the principal shear strain can be obtained using Eq. 3-4.

\[ \gamma_{\text{max}} = \sqrt{2\left((\varepsilon_i - \varepsilon_j)^2 - (\varepsilon_i - \varepsilon_k)^2\right)} \]  

(3-4)

Figures B-1 to B-6 present the shear strains versus force and relative displacement of the two segments. The maximum shear strain reached 40000 microstrains, which is approximately 15 times the yield strain in shear. It is worth mentioning that, this large post yielding shear strain was local and does not mean that the entire section of the pipe has reached this strain level.

The main purpose to plot the shear strain versus displacement was to investigate the effect of shear plastic deformations on total displacements. As marked in Fig. B-3 with an arrow, there is a change of slope in the strain-displacement curves in all cases. Before this point, displacement changes in proportion to the shear strains, that is to say shear yielding had a perceptible contribution to the total displacement. After that point the rate of increase in displacement was substantially higher; which means that other sources of deformations were contributing to the total displacement. This point was the border between part (1) and part (2) as explained in previous section. Major source of deformation after this point was the concrete bearing failure rather than shear yielding.

3.2.3.2. Normal Strains on the Pipe

The flexural strains on the pipe were monitored using fourteen unidirectional strain gauges (seven on each side of the pipe) that were installed on the pipe as explained in Chapter 2. These gauges are numbered SG20 to SG33. The measured strains for these
gauges are presented in Appendix B of this report in Figs. B-7 to B-18. Figure 2-27 shows the labeling of the gauges.

Depending on the depth of the strain gauge, different levels of flexural yielding were recorded. Table 3-1 shows the maximum strains that were recorded during each test. Note that the yield strain was 1650 microstrains. The maximum flexural plastic deformations were recorded on gauges SG22, SG23 on one side and SG29, SG30 on the other side of the pipe. These gauges were located at a depth of 2 in (50.8 mm) and 3.5 in (88.9 mm) in SP1/2P-A/B, and 1.75 in (44.45 mm) and 3.25 in (82.5 mm) in SP3P-A/B, respectively. This confirms the formation of flexural plastic hinges at these depths when specimens were under push loading. The maximum flexural strain was as high as 30 times the yield strain. The peak strains were less than the yield strain when the specimens were under pull loading. The maximum strain was recorded in PS2P-A/B that were built without inner spiral. It was due to the extensive cracking of the concrete which let the pipe bend more.

Many of the strain gauges broke before the ultimate loads were reached due to the enormous contact stresses between the pipe and the concrete. Figure 3-15 shows sample of strain gradient profile through the length of the pipe for PS1P-B under different loads or deformations levels.

3.2.3.3. Strains on Concrete Surface

Four strain gauges were installed on the surface of concrete to study the distribution of the stresses around the pipe. The strain plots are presented in Figs. B-19 to B-24. The
gauge labeling is presented in Fig. 2-27 for these gauges. The recorded value by SG37 was a measure of tensile strains on the side of the pipe. Under pull loading, the splitting strain reached approximately 4400 microstrains in PS1P-B while in PS2P-B it was as high as 30000 microstrains, which shows the significance of the inner spirals in controlling the concrete cracking.

Another important observation can be obtained from SG35 which measured the strains parallel to the loading direction. This strain gauge recorded strains as high as 11300 microstrains in PS1P-B, while the corresponding value was limited to 3100 microstrains before the concrete spalled in PS2P-B. This indicates that the inner spiral also increased the ductility of concrete under bearing stresses from the pipe.

3.2.3.4. Strains on Spirals

Strain gauges SG10 to SG15 were installed on inner spiral and SG16 and SG17 were installed on outer spirals as shown in Fig. 2-28. The results are plotted on Figs. B-25 through B-30. The inner spirals yielded under push loading but not under pull loading where the maximum loads were lower. The data confirms that the inner spirals mobilized and confined the compressed concrete. When inner spiral was absent, the strains were higher on the outer spirals.
3.2.4. Calculating the Equivalent Uniform Bearing Strength of Concrete

3.2.4.1. Literature Review

It is well known that the compressive strength of concrete increases due to confining transverse pressure. The same effect is expected for a small volume of concrete within a large body that the massive surrounding concrete provides confining effect small loaded area. Studies that address this issue are scarce. The available experimental data are mostly limited to the study of bearing strength of concrete against steel dowel bars. The diameter of bars is considerably smaller than the diameter of steel pipes (less than 1 in, 25 mm verses 12 in ~ 20 in, 300 mm ~ 500 mm). Furthermore, there is a substantial scatter in the empirical equations that are proposed by different investigators for concrete bearing strength on the side of bars. Researchers have suggested a bearing strength of concrete ranging from 1.45 to 5.00 times the concrete compressive strength [Soroushian, et al. 1988]. Soroushian et al. [1986] suggested an empirical equation (Eq. 3-5) relating the bearing capacity to the bar diameter, but the validity of this equation for pipes is uncertain because the ratio of bar diameter to the maximum aggregate size is relatively small, whereas for pipes the ratio is large.

$$f'_c = 37.6 \sqrt{\frac{f'_c}{d_b}}$$  \hspace{1cm} (3-5)

In another related study, Frosch [1999] used the bearing capacity equation (Eq. 3-6) that is suggested by ACI codes in section 10.17.1 [ACI 318]. Main focus of his research was investigating the behavior of pipe shear lugs that connect wall panels to concrete frames.
To address the existing gap in the current literature, push-off specimens were designed to obtain experimental data that is required for developing an empirical equation. In the push-off specimens, the diameter of the embedded pipe was chosen to be much larger than the maximum aggregate size.

There is no reliable method to directly measure the bearing stresses reacting on the pipe. An indirect method was employed to find the design equivalent uniform bearing stresses as is explained in next three sections.

3.2.4.2. Formulating the Behavior

Consider the free body diagram of an embedded pipe when it is cut at the level of one of the strain gauges as presented in Fig. 3-16. The equivalent uniform bearing stress is labeled as \( f_c^* \). The equilibrium of the moments about the bottom section results in Eq. 3-7. This equation implies that at any lateral load, if the associated moment at a certain depth is known, the equivalent uniform bearing stress can be estimated.

\[
f_c^* = 0.85 f'_c \sqrt{\frac{A_2}{A_1}} < 2 f'_c
\]  
(3-6)

\[
f_c^* = \frac{H(e + Y_s) - M}{0.5D_p Y_s^2}
\]  
(3-7)

\( f_c^* \): Equivalent uniform bearing stress against the pipe ksi (MPa)

\( H \): Lateral load kip (kN)

\( Y_s \): Depth of the strain gauge measured from concrete surface in (mm)

\( M \): The moment in the pipe at depth \( Y_s \) kip.in (kN.mm)
3.2.4.3. Moment-Curvature Analysis

Because the pipe-pin was a composite section of concrete filled steel pipe, a detailed moment-curvature analysis was required to relate the moments to strains on the surface of the steel pipe. The concrete inside the pipe is highly confined. The model proposed by Sakino and Sun [2004], which was developed specifically for highly confined concrete inside the steel tube columns known as concrete filled tube (CFT) columns, was used. Xtract section analysis software was used to do the analyses. Figures 3-17 and 3-18 show the moment-strain relations for the PS1/2P-A/B and PS3P-A/B, respectively. These curves are presented in terms of both tensile and compressive strains, because of the fact that the tensile and compressive strains are not equal for a specific moment. This is because of the concrete core that shifts the neutral axis towards the compressive side.

The moment-strain data were used to convert the measured strains to the corresponding moments at a desired depth and use the moment in Eq. 3-7 to determine the equivalent uniform bearing stress.

3.2.4.4. Equivalent Uniform Bearing Strength Against the Pipe

For each specimen, two pairs of strain gauges on the opposite sides of the plastic hinge were used to determine the moments at two depths. The associated lateral load for each
strain; \( H \), was obtained by using the measures load-strain results from the test. By substituting the moments and lateral loads in Eq. 3-7, \( f_c^* \) was determined for each strain gauge. Five post-yielding strains were used to calculate the equivalent uniform bearing strength. The average of these five equivalent uniform bearing stresses was defined as \( f_c^* \). Tables 3-2 and 3-3 show the detail results for each specimen. Negative and positive strains in these tables stand for tensile and compressive strains, respectively. Table 3-2 show the results associated with the negative strains (tensile strains) that were measured by SG22 and SG23, and Table 3-3 shows the results for positive strains (compressive strains) that were recorded by SG29 and SG30. The labeling of the gages is shown in Fig. 2-27. The tables show that smaller strains result in larger bearing stresses. The average of these five values was used for the \( f_c^* \), because this proved to be sufficiently accurate to estimate the capacity of the specimens, as will be discussed later.

Table 3-4 presents the summary of the results. The results show that the equivalent uniform bearing strength was approximately twice the uniaxial compressive strength of concrete. The average bearing strength for PS1P-A/B was \( 2.28f_c' \), while for PS3P-A/B this value was \( 2.35f_c' \). The reducing trend of bearing strengths by increasing the pipe size proves that for real size pipes (14 in ~ 20 in (335.6 mm ~ 508 mm)) the value of \( 2f_c' \) can be considered as an upper limit for the bearing capacity. This observation is compatible with the ACI design code [ACI, 2005] formulation as presented in Eq. 3-6 that limits the bearing strengths under base plates to twice the compressive strength of concrete.
An empirical equation was proposed based on the experimental results adopting the form of relationships that were developed by Soroushian [1986]. According to Soroushian, the bearing strength of concrete is related to the cubic root of the diameter and square root of concrete strength. Using the same approach, Eq. 3-8 was proposed in this study for the bearing strength of concrete using regression analysis of the results obtained from PS1P and PS3P as shown on Table 3-4. To develop this equation, a linear regression was done on the average ratio of \( \frac{f_c^*}{f'_c} \) versus \( 3\sqrt{D_p} \) data of Table 3-4 to find the slope and constant term (-1/3.35 and 2.95 in Eq. 3-8). In order to relate the bearing strength to concrete strength (as Eq. 3-5 implies), it was multiplied by ratio of \( \sqrt{f'_c/2.43} \) which is equal to one when the concrete strength on the test day (5.9 ksi, 40.7 MPa) is being used. It should be noted that the concrete strength obtained from this equation is an equivalent uniform stress and not the maximum bearing strength (Fig. 3-16). By comparing the results of PS1P and PS2P, it was recommended that, in the absence of inner spirals, 90% of the bearing strength obtained from this equation be used.

\[
f_c^* = \begin{cases} 
\frac{\sqrt{\frac{f'_c}{2.43}}}{2.95 - \frac{3D_p}{3.35}} f'_c \ (ksi) \\
\frac{\sqrt{\frac{f'_c}{6.4}}}{2.95 - \frac{3D_p}{9.85}} f'_c \ (MPa)
\end{cases}
\]

To verify this equation, it was used to determine the capacity of pipe-pins in push-off specimens by using Eqs. 3-9 and 3-10. The concept behind these equations is explained in detail in Ch. 6 of this report.
\[ L_1 = \sqrt{e^2 + \frac{2M_p}{D_p f_c^*}} - e \]  

(3-9)

\[ H_u = L_1 D_p f_c^* \]  

(3-10)

In these equations, \( L_1 \) is the nominal depth of the plastic hinge. Table 3-6 compares the analytical and experimental values. This table shows that the capacity was overestimated by an average of 5\% and 3\% for PS1P and PS2P, respectively; but the formulation underestimated the capacity by 6\% for the PS3P. This comparison confirms that the proposed formulation is reasonably accurate.

3.2.4.5. Actual Bearing Stresses Profile

The actual distribution of the bearing stress around the pipe is different from the uniform pressure discussed in previous sections. To obtain the real profile of the bearing stresses, another method was used based on the differential equation of a flexural member.

Assume that the pipe is a laterally loaded beam, in which the lateral load is the resisting stresses. The governing differential equation of a laterally loaded beam with a load intensity of \( D_p f_c^* (y) \) is:

\[ D_p f_c^* (y) = \frac{d^2M(y)}{dy^2} \]  

(3-11)

\( f_c^* (y) \): The bearing stress which act as an external load on the beam (pipe)

\( M(y) \): Moment in the pipe
The strain profile was translated to the corresponding moment profiles using the moment-curvature relationships as explained in previous sections. Then, the finite difference method was used to convert the moment to the equivalent lateral force and stress based on Eq. 3-11.

\[
(D_p f'_c)_{i} = 2 \left[ \frac{M_i - M_{i-1}}{Y_i - Y_{i-1}} - \frac{M_i - M_{i+1}}{Y_i - Y_{i+1}} \right]/(Y_{i-1} - Y_{i+1}) \]  
(3-12)

\( Y_i \) : The depth of the \( i^{th} \) strain gauge from concrete surface

\( M_i \) : Corresponding moment of the pipe at the location of the \( i^{th} \) strain gauge

As Eq. 3-12 implies, to determine the bearing stress at any depth, three reliable adjacent strain data were needed to find the second derivatives. Among the six specimens, only PS1P-B satisfied this condition. Fig. 3-19 shows a sample result with the bearing stresses normalized with the measured concrete compressive strength (\( f'_c \)). This figure shows that under large deformations, the pick bearing stresses can reach to 5.7 times the concrete compressive strength.

As explained earlier, the maximum bearing stress is different from the equivalent uniform bearing stress and cannot be used in Eq. 3-10 to estimate the lateral load capacity of the pipe-pins.
3.3. Pure Shear Specimens

3.3.1. Load-Deformation Response

The pure shear specimens were tested in double shear. Therefore, the load was divided by two to find the shear forces. The displacement was the average of readings from two Novotechnik transducers that were installed between the top and bottom rigid plates. Figure 3-20 shows the shear-deformation results for all three pairs of specimens.

Note that the results for each pair of specimens were very similar and there was no scatter in the data. The specimens were loaded until they were completely distorted. The curves for IPS-2A/B did not extend to failure because the capacity limit of the testing frame. However, significant yielding occurred in this pair of specimens before testing was stopped. Figures 3-21, 3-22, and 3-23 show one from each pair of specimens after the test. It is obvious that the main source of deformation was the shear yielding. It can be seen that all specimen were very ductile in shear.

3.3.2. Measured Shear Strains

Shear strains were monitored on one of each pair using a rosette strain gauge. Figure 3-24 shows extent of deformation of this rosette. Equation 3-4 was used to find the shear strain from the rosette composition. The shear load-shear strain data is plotted in Figs. 3-25, 3-26, and 3-27. The shear strain reached 150000 microstrains in IPS-1B and IPS-3B, but it did not go beyond 45000 microstrains in IPS-2B.
3.3.3. Yield and Ultimate Capacity of In-Filled Pipes in Pure Shear

3.3.3.1. Engineering Value of Yield and Ultimate Points

The main objective of performing experiment on pure shear specimens was to collect the necessary data for developing empirical equations for yielding and ultimate shear capacities of infilled pipes.

The load-deformation curves showed that the transition between elastic and plastic parts is very smooth and gradual and there is no well-defined yield point on these curves. The same is true for the ultimate load because the large ductility of the pipes in shear prevented pipe failure.

To determine the effective yield point, the offset method was used. A line was drawn parallel to elastic part, at a displacement of \( D_p/20 \) from the origin, and the intersection of this line with the force-displacement curve was used as the yield point. The ultimate point was assumed to be at a displacement that corresponds to a displacement ductility of eight. Figure 3-20 shows these points on the curves.

3.3.3.2. Empirical Equations for Yield and Ultimate Capacities

The shear capacity of a hollow steel pipe is found from Eq. 3-13, assuming that the effective shear area of the pipe is \( A_v = 2A_g/\pi \) and yield stress in shear is \( f_{ys} = f_y/\sqrt{3} \) [Choo, 2002].

\[
V_{y,\text{pipe}} = \frac{2A_g f_y}{\pi \sqrt{3}}
\]  

(3-13)
Adding another term to account for the effect of concrete inside the pipe [Restrepo, 2005], leads to Eq. 3-14 and 3-15 for the yield and the ultimate shear capacities. The coefficients of the second terms were obtained from the experimental data in the present study.

\[
V_{y,\text{infill}} = \frac{2A_e f_y}{\pi \sqrt{3}} + \begin{cases} 
0.47A_e \sqrt{f'_c} & \text{(ksi)} \\
1.23A_e \sqrt{f'_c} & \text{(MPa)}
\end{cases} \quad (3-14)
\]

\[
V_{u,\text{infill}} = \frac{2A_e f_u}{\pi \sqrt{3}} + \begin{cases} 
0.93A_e \sqrt{f'_c} & \text{(ksi)} \\
2.47A_e \sqrt{f'_c} & \text{(MPa)}
\end{cases} \quad (3-15)
\]

Table 3-6 presents the analytical versus experimental results. The calculated results are also marked on Fig. 3-20. The table shows that the calculated results were within 3% of the experimental results. The yield and ultimate stresses of the steel material used in these equations were obtained from the material test reports.

### 3.4. Two-Column Bent

The observations and results from the two-column bent experiment served two purposes. The first was to evaluate the performance of pipe-pin that was designed using the proposed design guideline. The second was to investigate the seismic performance of the concrete filled FRP tube column.

Experimental data that are based on measurements from the instruments that were discussed in Chapter 2 are presented in Sections 3.5.3 through 3.5.9. The sampling rate of data acquisition system was 128Hz for the first four runs, but in the last three runs, it
was increased to 256Hz to better capture a very high frequency fluctuation of loads under impact between the pipes and the cans.

3.4.1. Achieved Table Motions

Since the shake tables and the bent model are two separate systems that interact with each other, the achieved motions of the shake tables are highly dependant on the mass and stiffness of the bent. The software that drives the shake tables modifies the target motions during testing as an attempt to compensate for the response of the payload on the tables. However, the bent model in this experiment is highly nonlinear and relatively stiff and strong with respect to the tables, which made compensation of the motions difficult.

The input ground acceleration was factored by 0.1 to 1.9 in 0.3 increments, which led to total number of seven runs. The time coordinate of the input acceleration was compressed by a factor of 0.447 to account for the scale of the test model that was 0.2. Figures 3-28 through 3-34 show the comparison of the target pseudo acceleration spectrums of the target motions versus the achieved ones. The measured natural period of the structure before each run is indicated by the dashed line to identify the frequencies that were of concern.

In general, there was an acceptable match between the target and achieved table motions in a range of natural period plus/minus 0.1 second except for the second run in which the motion was overshot.
3.4.2. General Observations

3.4.2.1. Reinforced Concrete Column

The damage progression photographs for the reinforced concrete (RC) column are presented in two groups. The first group, which includes Figs. 3-35 through 3-36, shows the north and south faces of the bottom of the column. The second group (Figs. 3-40 to 3-43) shows the east and west faces of the column at the pipe-pin hinge area. The east and west faces were the faces at which most of cracking was expected at hinge area.

The initial flexural cracking at the bottom of the column began during run 2. During run 4, flexural cracking increased and spalling of concrete in the plastic hinge region was initiated on north face of the column. Additional concrete spalling was observed after run 5 on the south side of the column. During runs 6 and 7, there were major increases in flexural cracking and spalling on both sides of the plastic hinge zones. The spiral reinforcement was exposed on the bottom of the column in run 6. During run 6, the longitudinal reinforcement was exposed on both sides of the column base. Extensive X-shape cracking pattern was also observed on the column base as flexural cracks turned into shear cracks (Fig. 3-39). No significant core damage was observable after the last run, but the column underwent extensive damage to the cover concrete.

The top hinge area was also carefully monitored during the test. The first diagonal crack appeared on the west and east sides of the column during run 3. The second crack appeared on the west face of the column in the hinge area during run 6. The north and
south top edges of the column were also damaged during run 6 due to spalling of concrete as the throat gap was closed at large rotations.

3.4.2.2. Concrete Filled FRP Column

The damage progression photographs for FRP tube column are also presented in two groups for the bottom and top of the FRP tube column. Figures 3-44 through 3-47 show the north and south faces of the bottom of the FRP tube column, and Fig. 3-50 to 3-53 show the east and west face of the FRP tube column at the pipe-pin hinge area at the top.

No sign of damage was detected on the FRP tube column until run 7 during which the FRP tube ruptured. The tensile rupture of the FRP tube occurred at the north side. Figure 3-49 shows the close view of the ruptured fibers. The opposite side of the FRP tube buckled at the base of the column in an elephant foot shape as indicated by an arrow in Fig 3-47. The footing started to crack in a radial pattern at run 5 on the south side (Fig. 3-46) and the area adjacent to the FRP exhibited local failure due to bearing stresses during run 6 as shown in Fig. 3-48.

No damage was observed at the top of the FRP tube column in the pipe-pin hinge area.

3.4.3. Measured Load and Displacements

3.4.3.1. Displacement of Individual Columns and Hinge Slip

The lateral displacement of the cap beam was measured by two potentiometers that were attached to the east and west side of the mid height of the cap beam. The readings from
these two instruments were averaged and subtracted from the table displacements in order to calculate the relative displacement of the top of the bent.

The bent displacement was the summation of the column displacement and the horizontal slippage that occurred in the pipe-pin hinges between the column and the bent cap. Two Novotechnik transducers were installed at the north and south sides of each column along with two on the east and west faces to measure the slippage. The readings of the east and west transducers were not used because the attaching rod broke at early runs due to the impact shocks. Figure 3-54 shows the north and south Novotechnik transducers. A small slippage in the hinge is also visible in this figure and is highlighted by an arrow.

Comparing the readings of these two Novotechniks revealed a sudden jump when the friction capacity was exceeded. It appears that the large velocity of the column applied a displacement pulse to the middle rod of the Novotechnik and moved it backward. The rod was then returned to the initial position by the spring action of the instrument. As an example, these spikes are shown for run 6 in Fig. 3-55. The symmetry of the readings of these two Novotechnik is also noticeable. The data from the opposite Novotechnik transducers was used to manually remove the jumps and clean the data from these unreal spikes.

Figure 3-56 and 3-57 show the history of the hinge slip after correcting the data. These graphs show that the first sliding occurred during the third and second runs in RC column and FRP tube column, respectively. The maximum hinge slip reaches approximately 0.4 in (10 mm) in both columns.
Figure 3-58 and 3-59 show the column displacement histories. The maximum displacement reached 5.04 in (128 mm) and 4.98 in (126.5 mm) in the RC and FRP tube columns, respectively, which corresponds to approximately 8.4% drift. The maximum drift that was recorded for the bent was 9.1% including the hinge slip.

Figure 3-60 is a plot of the displacement of RC column versus FRP tube column. The figure shows that the columns moved in phase, as expected.

3.4.3.2. Load Distribution between Columns

It was explained in Chapter 2 that a load cell was placed in the middle of the bent cap beam to help measuring loads in each column. The middle load cell reading was the base shear of the FRP tube column. The base shear of the RC column was obtained by subtracting the readings of the middle load cell from the readings of the link load cell. Additional adjustments had to be made to account for the inertial force caused by the steel loading beam that was placed on top of the cap beam, the mass link, and other attachments.

Some modification was necessary to the raw data. Two sources of error existed in lateral load readings. The first was initial (residual) load in the middle load cell before starting the experiment. As was explained in Chapter 2, the cap beam was built in two segments and was post-tensioned to the middle load cell, using an approximately 200-kip (890-kN) prestressing force. Because the bent was an indeterminate structure, the slight shortening of the bent generated a force in the columns towards each other. The column reaction produced a tensile force in the middle load cell before data monitoring was started. The
load cells readings were zeroed at the beginning of the experiment. Therefore, it was necessary to estimate the magnitude of the tensile force and subtract it from the reading of the middle load cell. This force was calculated from an elastic SAP2000 model of the bent that was built to design the post-tensioning rods. The model showed that the shear in the columns due to the prestressing force was 2.5 kip (11.12 kN).

Another source of error was significant noise in the data due to the impact between the pipe and the can each time the friction force was exceeded and side gap closed. The impact energy propagated through the length of the bent cap as axial compressive waves and added a very high frequency noise to the load cell readings. Figure 3-61 shows a zoom in window on data obtained during run 6. Five impacts are detectable in this graph. The local maximum was as much as 80% larger than the real maximum of the smooth curve.

To smooth the data, several filtering methods were examined. Finally, the moving average method was selected as the most efficient one. This method is a time domain filtering technique that can remove the noise from data without distorting it. In this method the smoothed $n^{th}$ value is obtained from Eq. 3-16:

$$x_n = \frac{\sum_{i=a}^{a+\alpha} x_i}{2a + 1}$$  \hspace{1cm} (3-16)

2$a$ data points will be lost in this method from the beginning and end of the data series. $a$ was assumed to be equal to 5 to obtain a reasonable result without losing many data points. In other words, the amplitude of each point in the smoothed curve is the average
of 11 points, five before and five after. Figure 3-61 compares the smoothed and raw load cell readings for the three high-amplitude seconds of the sixth run.

Figures 3-62 through 3-64 present the force-displacement hysteresis curves of the bent, the RC column, and the FRP tube column, separately after removal of noise from the load and displacement data. It can be seen that the hysteresis curves were symmetric mostly because of the input earthquake waveforms and the resonant characteristics of the motion and the bent. The area within the hysteresis loops was larger in the conventional RC column in comparison to the FRP tube column indicating larger energy dissipation in RC column. Figures 3-65 and 3-66 show the backbone force-displacement curves, which are the envelopes of the hysteresis curves in the positive and negative displacement zones. The maximum load in the RC column was 40.5 kip (180.14 kN) and in the FRP column was 44.5 kips (197.9 kN). The hardening after yielding was more significant in the FRP column because the FRP tube remains elastic, while the steel reinforcing bars in the RC column yield.

The force is plotted against the slippage in pipe-pin hinges in Figs. 3-67 and 3-68 to find the friction resistance. The friction force varies at different cycles, with an average of approximately 20 kip (89 kN) for both pipe-pin hinges.

3.4.4. Dissipated Energy

The dissipated energy was measured by integrating the area enclosed by the force-displacement curves. This energy was calculated for pipe-pin hinges and the columns. The accumulated dissipated energies are presented in Figs. 3-69 to 3-71. It should be
noted that the main source of energy dissipation in the pipe-pin hinges was the friction damping as opposed to damping due to plastic deformations.

Out of 2275 kip.in (257028 kN.mm) total cumulative dissipated energy after seven runs, approximately 7.2% was dissipated at each pipe-pin hinge, 52.2% was damped by yielding of the RC column, and 33.4% was dissipated through the plastic deformations in the FRP tube column.

3.4.5. Measures Strains

As discussed in Chapter 2, and shown in the instrumentation plan of Figs. 2-63 through 2-64, strains were measured during the test at several longitudinal and lateral reinforcement bars in the columns, along with longitudinal, hoop, and shear strains on FRP tube. Strains on the pipe, spirals around the pipe-pins, the exterior can, and the longitudinal bars in the bent cap were also monitored during the test.

Strain gauges are generally reliable for strains of up to approximately 200000 microstrains (20 percent). While strain gauges are accurate instruments, cracks and interaction of aggregates and ribs on the bars with the cement paste can cause highly localized strains. Therefore, erratic measurements may be recorded during some tests. Some of the strain gauges or their wires broke during the test. For these gauges only the meaningful part of data is shown on graphs.
3.4.5.1. Conventional RC Column

3.4.5.1.1. Longitudinal Reinforcement

The strain gauge readings on the longitudinal bars in plastic hinge area of the conventional RC column are presented in Figs. C-1 through C-16. Almost all of the strain gauges that were installed on the longitudinal bars could record reliable data through the end of the experiment. Table 3-7 present the maximum and minimum measured strains for each run. Positive values in this table stand for compressive strains and negative values show tensile strains. The maximum strain of the longitudinal reinforcing bars reached 65800 microstrains in tension which is approximately 28 times the yielding strain.

Figure 3-72 shows the maximum and minimum strain profiles of the outermost northern longitudinal bar. The figure reveals that most of the plastic strain occurred in the lower 10 in (254 mm) portion of the RC column.

3.4.5.1.2. Transverse Reinforcement

The force-strain plots of the spirals in the plastic hinge area of the RC column are shown in Figs. C-17 to C-24. Table 3-8 summarizes the maximum measured strains. The maximum strain on the transverse reinforcement was 2176 microstrains which is slightly less than the measured yield strain of 2200 microstrains.
3.4.5.1.3. Steel Pipe-Pin

The strain hysteresis curves that were measured on different locations of the steel pipe in the RC column are presented in Figs. C-25 to C-34 including the longitudinal and shear strain recordings. Table 3-9 is the summary of the largest recorded longitudinal strains on the steel pipe. Note that negative strain indicates tension. Unexpectedly, the largest strains were recorded during run 5 in all of the gauges. This is due to the sharp jump caused by the impact between the pipe and the can. These jumps were not filtered because they were real and were not caused by local vibrations. The large strain peaks in run 5 indicates that in this run the impact was the largest.

The measured bending strains on the pipe were less than the yield strain of the pipe, which was 1800 microstrains. This indicates that the pipe-pin remained elastic during the seismic loading as intended. To measure the shear strains three strain components were needed. However, one or two of these gauges malfunctioned during early runs and no usable data was obtained from strain gauge rosettes.

3.4.5.1.4. Inner and Column Spiral in Pipe-Pin Hinge Area

Figures C-35 trough C-46 show the measured load strain relationships in the confining spirals at the hinge area. Table 3-10 shows the extreme values. It can be seen on this table that the maximum strain in the inner spirals was less than 1000 microstrains, and well below the yield strain of 2200 microstrains. Similar to measured strain on the pipe, the maximum strains on the spirals were recorded during run 5 when the largest impact took place.
In general, the measured strains were slightly larger on the inner spiral compared to the column spiral around the pipe. The strains reduced as the distance from the top of the column increased.

3.4.5.2. FRP Tube Column

3.4.5.2.1. Longitudinal Reinforcement

The strain gauge readings of the longitudinal bars in the plastic hinge area of the FRP tube column are presented in Figs. C-47 through C-53. Table 3-11 presents the maximum and minimum measured strains for each run. The maximum axial strain of the longitudinal reinforcing bars reached 48660 microstrains which is approximately 21 times the yielding strain. This shows that the longitudinal bars yielded and helped dissipate energy.

Comparing the strain hysteresis curves of the longitudinal bars of the RC column and FRP tube column indicates that the compressive strains were smaller in reinforcing bars of the FRP tube column. This is associated to the fact that the depth of natural axis was smaller in FRP tube column because the encased concrete had a much larger compressive strength compared to the core of the RC column. This was also because the cover concrete crushed in RC column and steel bars carried the extra load, while in FRP tube column the FRP shell helped charring compressive forces in addition along with confined cover concrete inside the tube.

Figure 3-73 shows the maximum and minimum strain profiles of the outermost southern longitudinal bar. The figure shows that the distribution of yielding is completely
different from the conventional RC column (Fig. 3-72) and that plasticity is spread over a larger portion of the column length. This effect in FRP tube columns was also observed by Mirmiran [Zhu, Mirmiran, Saiidi; 2006]. As opposed to the conventional column, the maximum tensile strains occurred in 5 in (127 mm) higher than the footing level, but the peak compressive strain profile was similar to that of the RC column. Also the strain on the longitudinal bars was larger in 5 in (127 mm) below the footing surface.

3.4.5.2.2. Longitudinal Strains on FRP Tube

Many of the strain gauges glued on the FRP tube malfunctioned during early runs because the flexural cracking of the resin broke the gauges. Horizontal flexural cracks damaged the longitudinal gauges more than the horizontal gauges. The recorded data to the extent that the gauges were operational are presented in Figs. C-63 to C-72. Table 3-12 also summarizes the maximum and minimum recorded strains.

The profile of the longitudinal strain on FRP tube for first three runs is resented in Fig. 3-74. The strain profiles are quite similar for tension and compression in low amplitude runs.

3.4.5.2.3. Hoop and Shear Strains on FRP Tube

Hoop strains on the tube were measured to understand the magnitude of hoop stresses that were generated because of the confining effect. Figures C-73 to C-78 show the hoop strain hysteresis curves. The maximum hoop strain reached 18220 microstrains at 1 in (25 mm) above the footing level. This strain is substantially higher than the maximum spiral strain of 2176 that was measured in the RC column.
Figures C-79 to C-82 show the shear strains recorded by the rosette gauges that were installed on the sides of the column.

3.4.5.2.4. Steel Pipe-Pin Strains

The measured strain hysteresis curves for the steel pipe in the FRP tube column are presented in Figs. C-79 to C-92. Table 3-13 lists the peak recorded longitudinal strains on the steel pipe.

Excluding strain gauges FP8 and FP7 which recorded unreliable data, the measured bending strain of the pipe was less than the yield strain of the pipe steel, confirming that the pipe-pin remained elastic, as intended in design. The maximum measured shear strain on the pipe was 500 microstrains which is approximately one-fifth of the yield strain in shear.

3.4.5.2.5. Inner Spiral and Hoop Strain on FRP in Pipe-Pin Hinge Area

Figures C-93 through C-102 show the hoop strains of FRP tube and strains of inner spiral at the hinge area. Table 3-14 shows the extreme values. It can be seen from the table that the maximum strain in the inner spiral around the pipe was less than 1000 microstrains. Similar to the RC column, the spirals at larger depths registered smaller strains.

In general, the measured strains were larger on the FRP tube compared to the inner spiral, as oppose to the RC column in which the strains on inner spiral were larger than what of column spirals. This is because of the smaller module of elasticity of FRP compared to
steel (2000 ksi (13800 MPa) vs. 29000 ksi (200,000 MPa)), which made the FRP tube stretch more to generate comparable hoop forces.

3.4.5.3. Cap Beam Strains

3.4.5.3.1. Longitudinal Bars

Figures C-103 to C-110 present the force-strain relationships in the longitudinal bars of the bent cap beam. The very small strains means that little bending took place in the bent cap under lateral loading.

3.4.5.3.2. Steel Cans

The hoop strains recorded on the steel can are presented in Figs. C-111 to C-114. The strain gauge which was installed on the north side of the can recorded large strains. At this location the steel pipe came into contact with the can and led to local yielding of the can.

The other strain gauge which was installed on the side of the can recorded small strains.

3.4.6. Strain Rates

The measured strain rates were quite larger than what usually is expected in the similar shake table tests, the main reason was the characteristics of the applied ground motion. Figure 3-75 shows a sample of the measured strain rates for strain gauge C31 (longitudinal reinforcement gauge in the plastic hinge zone of the RC column) in the high-amplitude portion of run 6. Also shown in this figure is the strains amplified by a
factor of 10 to show the strain trends. This figure shows that the column underwent four full cycles between 59 sec and 61.5 sec which created very large velocities and correspondingly large strain rates. Another important aspect of this figure is the very sharp high frequency spikes upon impact between the pipes and the cans. The strain history did not show the impact times but strain rate history clearly indicated that.

The strain rate of concern was the average between strains equal to half of yielding to yielding (1200 to 2400 microstrains), because this is the rate that is used is the method to estimate the strain rate effect on the steel material properties that is presented in Chapter 4. Figs. 3-76 and 3-77 show the variation of the average rates plotted verses the maximum recorded strains during each run for the longitudinal bar gauges. Both of these figures show a general increase in the rate when the local maximum strains increased.

3.4.7. Measured Curvatures

3.4.7.1. RC Column

The curvatures and bond-slip rotation were measured by eight Novotechnik transducers that were installed on the columns as shown in Fig. 2-65. The rotation was calculated by dividing the difference of the readings of the two opposite transducers at each level by their distance.

Figures 3-78 shows the hysteresis curves of the rotation at the lower most level which is a measure of the bond slip rotation due to yield penetration of the longitudinal bars inside the footing. Figures 3-79 to 3-81 present the curvatures at the first, second, and third
levels of instrumentations from the footing, respectively. The curvature peaked at the bottom of column, as expected.

The profile of the maximum rotations that occurred between each two level of Novotechniks is presented in Fig. 3-82 for the last four runs.

3.4.7.2. FRP Tube Column

Figures 3-83 shows the load-rotation hysteresis curves at the lower most level which is a measure of the rotation of the embedded part of the FRP tube in the footing. The rotation at the base of the FRP tube column was comparable with the bond-slip rotation in the RC column.

Figures 3-84 to 3-86 present the curvatures at the first, second, and third levels of instrumentations on FRP tube column from the base, respectively. Several of the threaded rods that were used to attach the Novotechniks transducers broke during strong motions. The figures present the usable part of the data that was recorded during the test.

The maximum rotation profile of the column is presented in Fig. 3-87 for runs 2 through 6. There was no concentration of rotation along the length and the plastic hinge was relatively long, but the rotations gradually decreased at upper elevations. This figure also shows that most of the rotation came from the embedded part of the FRP tube column inside the footing.
3.4.8. **Pipe-Pin Hinge Rotations**

One of the important data that was measured during the test was the relative rotation between the top of the columns and the cap beam. This was done by using two Novotechnik transducers at the north and south faces of each column. Figures 3-88 and 3-89 present the hysteresis curves for the top rotation of the columns.

The maximum measured rotation was approximately 0.08 radians which is comparable to the maximum column drift ratio of 8.4%. The proposed design method assumes that the expected drift and pipe-pin rotation are the same, and the assumption was confirmed by this test data. The asymmetric response that can be seen in Figs. 3-88 and 3-89 is due to the fact that one of the Novotechniks reached the maximum stroke.

3.4.9. **Axial Load Variation and Vertical Displacements**

The rotation of the columns lifted the cap beam up and increased the axial loads on columns. The axial load rams were attached to an accumulator to minimize the fluctuation of the axial loads during the motion, but still some instability was observed because of the small diameter of the connecting hoses that did not allow the surcharge hydraulic oil flow to the accumulator fast enough.

Prior to the start of the test, a 48-kip (213.5-kN) axial load was applied on each column using four high strength threaded rods, but for the last three runs, there was a concern regarding exceeding the capacity of the pancake load cells that were measuring the axial loads; therefore, the axial load was reduced to 42 kips (186.8 kN) prior to runs 5 and 6 and to 38 kips (169 kN) prior to the seventh run.
Figures 3-90 and 3-91 show the axial load history for each column. The axial load reached 64 kip (285 kN) and 75 kip (334 kN) for the RC column and FRP tube column, respectively. The effect of the extra axial load was studied by conducting moment-curvature analyses of the columns. It was found that the moment increased by 1.5% and 2.4% for the RC and FRP column, respectively, which was negligible. In the analytical studies, the measured axial load histories were used, rather than the target axial load.

The vertical movement history of the cap beam ends are presented in Figs. 3-92 and 3-93. The cap beam was pushed up by 0.8 in (20.3 mm) at the maximum lateral deformation.

3.4.10. Dynamic Characteristics

Natural period of the bent increased by increasing the level of damage. The period of the structure was calculated using the response of the structure to the white noise after each run. The Fast Fourier Transform (FFT) of the structure response to the white noise motions was calculated and the natural period was extracted by finding the peak values.

Prior to testing, the natural period of the structure was 0.245 sec and increased to 0.714 sec after run 7. Figure 3-94 shows the changes in the natural period during the test versus the maximum drift that was recorded at each run.

Figure 3-95 show the variation of the damping ratios that were obtained using the FFT curves [Chopra, 2005]. The damping was very large in early runs (approximately 30%), but it was reduced to 14% after run 3, but it increased to 20% after run 7. The large damping of early runs can be a result of friction in the mass-rig joints.
3.4.11. Evaluation of Pipe-Pin Connections

As mentioned earlier, the main purpose of testing the two-column bent model was to evaluate the performance of the pipe-pin hinges that were designed using the proposed design method, which is presented in Chapter 6.

As far as the integrity of the structure and mitigation of damages in the hinge area was concerned, the test confirmed that the design guideline is safe and reliable. There was no significant damage at the hinge area. The pipe-pin hinges withstood the maximum plastic shear of the columns. Furthermore, the strain data also confirmed that the steel pipe and the surrounding spirals remain elastic and satisfied the requirement of “capacity protected” elements.

After the test, the cap beam was removed and the condition of the pipe-pin connections was examined. The pipes were perfectly straight with no sign of damage to them as Figs. 3-96 and 3-97 show. The edge of the RC column was chipped off due to contact between the column and the cap beam (Fig. 3-98). This damage was expected because the thickness of the hinge throat was too small to prevent gap closure. Nevertheless, this damage is considered to be minor and is repairable after the earthquake. Two thin cracks formed on the sides of the pipe as marked by solid arrows in Fig. 3-97. The extension of these cracks was detected on the column side surface during the test as explained in previous sections. These cracks appear to be insignificant. No cracking was observed on the FRP tube column around the pipe-pin hinge.
Exterior cans were slightly deformed in the contact areas with the pipes, as shown in Fig. 3-99. The edges of the bearing areas were ground due to large number of load cycles. Figures 3-100 and 3-101 show the status of the hinge throat after the test. Less earthquake damage is expected in hinges in actual bridges because the number of full cycles that they experience during major earthquakes is usually less than three, while the number of times the friction force was exceeded in the test was approximately 20 times. On the other hand, it should be noted that hinges in real bridges may be subjected to temperature variations that could lead to forces exceeding the friction resistance numerous times during the life of a bridge.

Overall, the hinges were found to meet the performance objective remaining elastic and essentially damage free. The status of the hinges after undergoing seven runs of a demanding earthquake was satisfactory, and the hinges were able to carry the weight of the superstructure, which is required by design codes.

3.4.12. Evaluating the Performance of FRP Tube Column

The shake table testing of the FRP tube column was the first of its kind. This system showed a great potential as a ductile load carrying structural element. No damage was noticeable to the column before the last run in which the tube fractured. This is a clear advantage over the RC column, in which the unconfined concrete began to spall during run 4.

There were some concerns about the full composite action of the FRP tube column because the inner surface of the FRP tube was extremely smooth. Visual examination of
the column after the test showed that the concrete core have protruded out of the FRP tube by approximately 0.2 in (5 mm) as demonstrated in Fig. 3-102. On the other hand, there was an acceptable compatibility between the strains recorded on the FRP tube and on the longitudinal bars. Figure 3-102 show the strains recorded at the strain gauges at approximately the same levels. The strains started deviating from each other after the strain on FRP tube reached 0.004, but the deviation was not significant. The data shows the results up to run 4, during which the top of the pier reached a drift ratio of 4.3%. As indicated before the strain gauges on the FRP tube malfunctioned in subsequent runs. It is hence concluded that slippage between the concrete and the FRP tube occurred during the last 3 runs.

As mentioned in previous sections, the failure of the FRP tube column during run 7 was due to the tensile rupture of the outermost fibers and local buckling of the tube in the compression side. Close-up views of the damaged faces are shown in Figs. 3-104 and 3-105.

3.4.13. Calculating the Plastic Hinge Length in FRP Tube Column

It was explained in previous sections that the plastic hinging took place over a longer length in the FRP column than that of the RC column according to the measured strains and curvatures. The theoretical plastic hinge length, $l_p$, was calculated based on the measured plastic displacement at the top of the column and compared with the theoretical plastic hinge length of an equivalent conventional RC column.
Figure 3-106 shows the bilinear simplification of the measured backbone curve for the FRP column. The yield and ultimate displacements were 0.443 in (11.25 mm) and 5.41 in (137.41 mm), respectively. The difference is the plastic displacement that is 4.97 in (126.24 mm). The moment-curvature analysis of the FRP column section is shown in Fig. 3-107. The yield and ultimate curvatures were 0.000421 in\(^{-1}\) (1.66e-5 mm\(^{-1}\)) and 0.0054 in\(^{-1}\) (2.13e-4 mm\(^{-1}\)), respectively. The theoretical plastic displacement can be calculated as:

\[
\delta_p = \left( L_c - \frac{l_p}{2} \right) (\varphi_u - \varphi_x) l_p
\]  

(3-17)

\(L_c\) : Length of the column

Substituting the numerical values and solving Eq. 3-17 for \(l_p\), results in plastic hinge length of 20.2 in (513.1 mm). Priestley formula for plastic hinge is:

\[
l_p = 0.08L_c + 0.15d_b f_y
\]  

(3-18)

\(d_b\) : Longitudinal bar diameter  \(\text{in}\)

\(f_y\) : Yield strength of longitudinal bars  \(\text{ksi}\)

Using \(f_y = 67 \text{ksi}\) and column length of 59.5 in, Eq. 3-18 results in a plastic hinge length of 9.7 in (249 mm) for a similar RC column. The information shows that the plastic hinge length in FRP tube columns is approximately twice that of the conventional RC column.
4. **ANALYTICAL STUDIES**

4.1. **Introduction**

The focus of this chapter is on the analytical modeling of the specimens with the objective of establishing the validity of the modeling assumptions based on the correlation between the analytical and experimental results. Three types of analytical studies were performed, each for a different purpose. The analytical models were: a simple stick model for push-off specimens, detailed finite element model of push-off specimens and PF-1, and an analytical model of the two-column pier.

4.2. **Simple Stick Models for Pipe Shear Keys**

4.2.1. **Introduction**

As stated in previous chapters, when a steel pipe acts as a shear key, its failure may be due to either concrete bearing failure or pure shear failure of the infilled pipe. These failure modes may occur when the pipe is surrounded by a relatively large body of concrete that eliminates the possibility of tensile fracture of concrete.

The concrete around the pipe can be modeled using a series of unidirectional springs perpendicular to the steel pipe. The steel pipe itself can be modeled using a beam element with possible flexural and shear nonlinearities. This model was used to simulate the load-deformation response of the push-off specimens and the full-scale prototype of pipe shear keys. SAP2000, version 11 [Computers & Structures, 2007] nonlinear
structural analysis package was used for this purpose. The push-over method was put into practice to obtain the load-displacement curves.

4.2.2. Modeling Details

4.2.2.1. Components of the Model

Figure 4-1 shows a sketch of the stick model. Three types of uncoupled lumped plastic hinges were used in this model: axial, rotational, and shear. Nonlinear axial springs that are shown by the zigzag springs in this figure modeled the surrounding concrete. The rotational plastic hinges that are shown as hollow circles in the figure modeled the flexural yielding of the pipe and the shear plastic hinges, the hollow squares, modeled the plastic shear deformations of the pipe.

The push-off specimens did not include the steel can and the associated gap. However, the can and the gap need to be included in pipe-pin shear key details. Figure 4-2 shows the proposed model for pipe shear keys that incorporates the gap elements. The axial springs are included to model the possible contact between the pipe and the can and the resulting moment that will be generated in the pipe by the force couple.

Ignored in this model is the friction force at the concrete-steel interface to simplify the modeling.

4.2.2.2. Concrete Nonlinear Springs

The bearing behavior of concrete against the steel pipe was replicated using a series of uncoupled nonlinear axial springs representing the tributary area of the concrete. Three
parameters were required to define the behavior of these axial springs: elastic stiffness, yield force, and ductility.

Other researchers have suggested some values for the equivalent stiffness of the concrete compressed against steel dowel bars. These values are in fact subgrade modulus multiplied by the projected area of the bar, similar to theory of beam on elastic foundation. A large scatter exists in data because of different approaches by which the equivalent stiffness of the subgrade has been calculated. Many researchers have assumed that the only source of the nonlinearity is the flexural yielding of dowel bars while concrete remains elastic. This assumption leads to very small stiffness.

In this study, Eq. 4-1 was proposed for the subgrade modulus based on Soroushian’s [1987] relationship that assumes subgrade modulus is proportional to the square root of concrete strength and the bar diameter (pipe diameter herein) to the power of -2/3. Constant coefficient of this equation was found by fitting the analytical elastic stiffness and measured ones from push-off experiments.

\[
k^* = \begin{cases} 
1000 \frac{\sqrt{f_{c}'}}{D_p^{2/3}} & \text{(ksi)} \\
893 \frac{\sqrt{f_{c}''}}{D_p^{2/3}} & \text{(MPa)} 
\end{cases}
\]  

(4-1)

In this equation \( k^* \) is in ksi/in or N/mm\(^2\)/mm. To find the elastic stiffness of the springs \( k^* \) should be multiplied by the projected area of tributary length of the pipe that the axial spring represents. When the inner spiral around the pipe was not present (in PS2P-A/B), 90% of the value from Eq. 4-1 was used as the subgrade modulus.
To find the yield strength of the axial springs, the empirical equation that was proposed in Chapter 3 (Eq. 3-8) was utilized. To find the ductility, two values were suggested based on experimental data; one for near surface concrete axial spring and the other for deep concrete springs according to the fact that the concrete at larger depths is more ductile because of larger confinement effect. The ductility of the springs located at less than $D_p / 4$ from the surface was assumed to be 2.2, but deeper springs were assumed to have a ductility capacity of 16. These values were based on the observed performance of the test models. $D_p / 4$ was approximately the measured depth of concrete spall in the push-off specimens.

In SAP2000, nonlinear rigid-plastic hinges only model the plastic portion of deformations. Total deformation of the nonlinear spring is summation of the elastic and plastic deformations. Therefore, the axial rigid-plastic hinge was assigned to a linear frame element with the stiffness value that was explained earlier. This combination produces the required elasto-plastic behavior. Fig. 4-3 shows the behavior of the rigid-plastic axial springs. Table 4-1 shows the properties of the axial springs that were used to model the push-off specimens.

4.2.2.3. Flexural Plastic Hinges

The pipe was modeled using a linear beam element. A series of rigid-plastic hinges were defined at different depths to capture flexural yielding of the steel pipe. Properties of the hinges were determined using moment-curvature analysis of the concrete-filled steel pipe. Details of the moment-curvature analysis are presented in Chapter 2.
By subtracting elastic curvature from total curvature and multiplying that by the tributary length of the pipe, “moment-plastic rotation” curves were obtained. Figure 4-4 shows the rigid-bilinear behavior that was fitted on the moment-plastic curvature curves. This rigid-bilinear model was used to define the nonlinear rotational hinges.

4.2.2.4. Nonlinear Shear Hinges

Equations 3-14 and 3-15 in Chapter 3 present the yielding and ultimate shear capacities of the infilled steel pipes. The associated yield and ultimate deformations were also explained in Chapter 3. Based on the experimental results, the load-deformation curve of Fig. 4-5 was used for the shear plastic hinges. Shear span ratio in the pure shear tests was $D_p / 2$, and this figure shows the values of shear plastic deformations over the length of shear span. Therefore, the load-plastic deformation curves were modified for length of the segment that was represented by the plastic shear hinge. The shear hinges were defined at the gap segment and the segments close to the concrete surface (Fig. 4-1).

4.2.2.5. Gap Element

The built-in gap model in SAP2000 was used to model effect of side gap between the pipe and the can. The gap thickness was defined according to the prototype pipe-pin detail (0.5 in, 12.7 mm). The gap model was a compressive gap that transferred the forces only when the gap was closed. The gap elements were defined at the end of the axial springs that were used to model the concrete over the length of the steel can.
4.2.3. **Analytical Results for the Push-Off Specimens**

The analytical models of push-off specimens were subjected to an incremental displacement at the point of loading (Fig. 4-1) until failure. Fig. 4-6 shows the distribution of plastic hinges after extensive yielding occurred. The shear hinges (shown in the middle) yielded at very early stage of loading but shear yielding did not spread. This was in agreement with stage (1) of the observed response as was explained in Section 3.2.2. Post yielding behavior and ultimate lateral capacity of the connection were controlled by flexural yielding of the steel pipe along with failure of concrete in bearing (the plastic deformations on axial springs of Fig. 4.6). Analytical mode of failure and sequence of damages was the same as the experimental observations.

Fig. 4-7 compares the analytical and experimental load-displacement curves. A good level of accuracy was obtained as far as the ultimate capacity and initial stiffness are concerned. The maximum capacities from the test for PS1P, PS2P, and PS3P were 130.2 kip (578.2 kN), 120.9 kip (538 kN), and 71.9 kip (320 kN), respectively. The corresponding values from the analysis were 125.5 kip (558 kN), 122.1 kip (543 kN), and 66.36 kip (295 kN), respectively that means the maximum error was of approximately 8%.

4.2.4. **Analytical Results for the Prototype Shear Keys**

After the analytical model was verified using the experimental data, the model was used to study the behavior of a full scale prototype pipe shear-key using SAP2000 [Computers & Structures, 2007]. The model of the prototype incorporated the gap elements in
addition to other plastic hinges to replicate the side gap between the pipe and the can as shown in Fig. 4-2.

Diameter and thickness of the steel pipe were 14 in (355.6 mm) and 1.25 in (31.75 mm), respectively. Embedded length and protruded length of the pipe were assumed to be 60 in (1524 mm) and 24 in (610 mm), respectively. Yield and ultimate stress of the steel pipe were assumed to be 55 ksi (380 MPa) and 66 ksi (455 MPa), respectively. Concrete strength was 5 ksi (34.5 MPa) inside and outside the pipe. This geometry was the same as Caltrans “Deign-A”. Analyses were carried out for two gap thicknesses of 0, and 0.5 in (12.7 mm). The associated load-displacement responses are plotted in Fig. 4-8. The pattern of yielding is presented in Figs. 4-9 and 4-10. There was a basic difference between the performances of the shear keys with two different gap thicknesses. When the gap thickness was zero, shear yielding was dominant and the lateral capacity of 1260 kip (5605 kN) was achieved. On the other hand, when the pipe was allowed to freely rotate inside the can by providing the 0.5 in (12.7 mm) gap, shear yielding occurred at the beginning similar to what was observed in push-off specimen and was immediately followed by flexural yielding of the pipe. The ultimate capacity was controlled by the bearing failure of concrete as shown in Fig. 4-10. In this case, the maximum load reached to 1090 kip (4848.3 kN) that was 13.5% lower than the capacity of the case with no gap.

In the first case, the connection did not perform as a moment free hinge and quite large moments were generated in the connection due to fixity of the pipe inside the can. Fig. 4-9 illustrates another important fact that even in pure shear yielding; the penetration of the
plasticity causes bearing failure in concrete to a depth of approximately 2 in (50 mm) on each side.

The analytical results showed that although shear yielding initiated nonlinear behavior of the connection, it was the flexural hinging of the pipe and the bearing failure of concrete that limited the lateral load capacity of the shear key. Therefore, the pure shear failure mechanism assumption overestimates the lateral capacity of the pipe shear keys.

4.3. Finite Element Modeling

4.3.1. Introduction and Scope

Basic characteristics of behavior of the pipe-pins hinges were understood through the experiments, as was explained in the last chapter. However, the detail comprised many parameters that needed to be studied to before developing the design method for pipe-pin hinges.

To model behavior of pipe-pin details under lateral loading, a sophisticated model that includes all the material, contacts, and large displacement nonlinearities was required. ABAQUS v6.7 [Hibbit, 2007] is a powerful FE packages that can serve this purpose. This FE package has been extensively used in many similar investigations such as study of bearing strength of concrete under steel base plates [Escobar-Sandoval et al., 2006] and study of behavior of headed studs [Lam et al., 2005], and parametric simulation of a new sheet-type shear connectors [Fink et al., 2006].
There are, however, many crucial unknowns to be addressed before starting FE simulation of an elaborated detail similar to pipe-pin hinges. In the case of the FE model used in this study the issues to be concentrated on were: choosing static or dynamic solution, defining control and stopping criteria, introducing material parameters especially for concrete, choosing element types, optimizing mesh pattern, defining boundary condition, selecting method of application of the load, using proper contact and friction models between surfaces, modeling reinforcement, and etc. FE analyses results are sensitive to the modeling assumptions and the input parameters.

The ABAQUS program was first used to model the push-off specimens. Extensive try and error process was involved to address convergence issue, rate of loading, material models, contact models, and reinforcement models. PF-1 (Sec. 3.4.3) [Doyle, 2009] was modeled following the same modeling method as that used for push-off specimens.

4.3.2. Modeling Method

4.3.2.1. Material Model

4.3.2.1.1. Concrete Material

ABAQUS/Explicit FE package has two types of nonlinear concrete material models: “Brittle Cracking” model and “Concrete Damage Plasticity” (CDP). The first one assumes a linear elastic behavior for the compressive behavior of concrete. This model is designed for cases in which concrete cracking controls the behavior. The Concrete Damage Plasticity (CDP) model includes nonlinearities in compression as well as tensile cracking. The latter one was used in this study.
The CDP model is a continuum, plasticity-based, damage model for concrete that is intended mainly for analysis of reinforced concrete structures subjected to monolithic, cyclic, and/or dynamic loading under confining pressures less than five times compressive strength of concrete. The element model assumes two failure mechanisms: (1) tensile cracking and (2) compressive crushing of the concrete material. It uses isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity to represent the inelastic behavior of concrete [Hibbit, 2007]. The evolution of yield surface is controlled by two hardening variables, $\tilde{\varepsilon}_t^{pl}$ and $\tilde{\varepsilon}_c^{pl}$, linked to failure mechanisms under tension and compression loading, respectively.

The model assumes that the uniaxial tensile and compressive responses of concrete are characterized by damaged plasticity, as shown in Fig. 4-11. Under uniaxial tension the stress-strain response follows a linear elastic relationship until the tensile failure stress, $\sigma_{tu}$, is reached. The failure stress corresponds to the onset of micro-cracking in the concrete material. Beyond the failure stress, the formation of micro-cracks is represented macroscopically with a softening stress-strain response, which induces strain localization in the concrete structure. Under compression the response is linear until the initial yield, $\sigma_{cu}$, is reached. In the plastic zone, the response is typically characterized by stress hardening followed by strain softening beyond the ultimate stress, $\sigma_{cu}$. This representation, although somewhat simplified, captures the main features of the response of concrete. It is assumed that the uniaxial stress-strain curves can be converted into stress versus plastic-strain curves. Damage under cyclic loading in CDP material is represented by reducing the slope of unloading and reloading curves. Considering the
fact that push-off specimens were loaded monolithically; and that the cyclic loading of
PF-1 was only during small loading amplitude, no material damage was defined for the
cement models in this study.

Two options are offered for modeling the tensile softening behavior of concrete. The
tensile material behavior can be modeled using either stress-strain or stress-displacement
relationship. Displacement in the latter case represents the crack width. When stress-
strain relationship is used, the smaller the size of element the narrower the crack while by
increasing the element size, the crack width increases (because crack width is equal to
tensile strain multiplied by length of the element). This observation is in contrast
physical behavior, for instance, in a concrete specimen developing a single tensile crack;
the displacement across the crack should be independent of the specimen length.
Accounting for this, the stress-displacement relationship was used for concrete in this
study.

The Hillerborg's [1976] fracture energy concept was used to determine the energy
required to open a unit area of crack, $G_f$, using “brittle fracture mechanics”. The area
under the stress-displacement curve is equal to the fracture energy. There are several
recommendations for value of $G_f$ and shape of tensile stress-displacement curves based
was assumed for the stress-displacement response of concrete in tension [Roesler, 2007]
as shown in Fig. 4-12. In this reference the initial fracture energy ($G_f$) was defined as
the area of the shaded part of Fig. 4-12. Equations 4-2 and 4-3 are used to determine
the crack widths \( w_i \) and \( w_f \).

\[
\begin{align*}
    w_i &= \frac{2G_f}{f'_t} \quad \text{(4-2)} \\
    w_f &= \frac{2}{\alpha f'_t} \left[ G_F - (1 - \alpha)G_f \right] \quad \text{(4-3)}
\end{align*}
\]

\( f'_t \): The standard tensile cracking stress

\( \alpha \): Assumed 0.25, based on Roesler [2007]

\( G_F \) and \( G_f \) were assumed to be 1.9e-3 kip/in (3.33e-4 kN/mm) and 7.4e-4 kip/in (1.29e-
4 kN/mm), respectively for normal strength concrete.

For compressive behavior, several models have been proposed by different researchers
[Babu, 2005]. Based on a review of these models, the Popovics [1973] model was used
in this study. In this model the compressive stress-strain relationship of concrete is
determined from:

\[
    f'_c = \frac{f'_c x r}{r - 1 + x'} \quad \text{(4-4)}
\]

Where:

\( f'_c \): is the compressive strength of the concrete

\[ x = \frac{\varepsilon_c}{\varepsilon_{co}} \] that, \( \varepsilon_{co} \) is the strain at maximum concrete stress of \( f'_c \) (assumed 0.002)

\[ r = \frac{E_c}{E_c - E_{sec}} \] where \( E_c = \begin{cases} 5000 \sqrt{f'_c} \quad \text{(ksi)} \\ 10500 \sqrt{f'_c} \quad \text{(MPa)} \end{cases} \) and \( E_{sec} = \frac{f'_c}{\varepsilon_c} \)
Note that if standard concrete cylinder is to be modeled using this curve with $\sigma_{co} = f_c'$, FE simulation of the concrete cylinder would show a different concrete strength than $f_c'$. The same is true for the split cylinder test result. Therefore, the values of $\sigma_{so}$ and $\sigma_{co}$ in the material model are different from $f_c'$ and $f_t$, respectively. To determine these two values a concrete cylinder was modeled in ABAQUS and in compressive and split cylinder test. The values of $\sigma_{so}$ and $\sigma_{co}$ were tuned in the way that the analytical compressive and tensile stresses match with $f_c'$ and $f_t$ that were obtained from the concrete tests. As explained earlier, only the plastic part of the strain was introduced to the program for tension and compression as defined in Eq. 4-5. The plot of uniaxial behavior of concrete is shown in Fig. 4-13.

$$\tilde{\varepsilon}^p = \varepsilon - \frac{\sigma}{E}$$  \hspace{1cm} (4-5)

The CDP model assumes a nonassociated potential plastic flow. The flow potential, $G$, that is used for this model is Drucker-Prager hyperbolic function of Eq. 4-6:

$$G = \sqrt{(\varepsilon \sigma_{so} \tan \psi)^2 - q^2} - p \tan \psi$$  \hspace{1cm} (4-6)

$\psi$: The dilation angle measured in $p-q$ plane at high confining pressure

$p$: Hydrostatic pressure stress ($-\frac{1}{3} \sigma_{ii}$)

$q$: Equivalent Mises stress $\sqrt{\frac{3}{2} s_i s_i}$, where $s_{ij} = \sigma_{ij} - \delta_{ij} p$

$\varepsilon$: Flow potential eccentricity, default value is 0.1
The dilation angle of $37^\circ$ was used in the analysis based on the literature [Fink et al., 2006].

The material model makes use of yield function proposed by Lubliner et al. [1989], taking into account the modifications proposed by Lee and Fenves [1998] to account for different evolution of strength under tension and compression. The evolution of yield surface is controlled by hardening variables, $\bar{\varepsilon}^{\text{pl}}_t$ and $\bar{\varepsilon}^{\text{pl}}_c$. The yield function takes the form of Eq. 4-7 in terms of effective stress.

$$F = \frac{1}{1-\alpha} \left( q - 3 \alpha p + \beta (\bar{\varepsilon}^{\text{pl}}_c) \bar{\sigma}_{\text{max}} + \gamma \bar{\sigma}_{\text{max}} \right) - \bar{\sigma}_{\text{c}} (\bar{\varepsilon}^{\text{pl}}_c) = 0 \quad (4-7)$$

Where:

$$\alpha = \frac{(\sigma_{bo}/\sigma_{co}) - 1}{2(\sigma_{bo}/\sigma_{co}) - 1}; \quad 0 \leq \alpha \leq 0.5$$

$$\beta = \frac{\bar{\sigma}_{\text{c}} (\bar{\varepsilon}^{\text{pl}}_c)}{\bar{\sigma}_{\text{c}} (\bar{\varepsilon}^{\text{pl}}_t)} (1 - \alpha) + (1 + \alpha)$$

$$\gamma = \frac{3(1 - K_c)}{2K_c - 1}$$

$\bar{\sigma}_{\text{max}}$: The maximum principal effective stress

$\sigma_{bo}/\sigma_{co}$: Ratio of initial biaxial compressive yield stress to initial uniaxial compressive yield stress (default value is 1.16)

$K_c$: Ratio of the second stress invariant in tensile meridian to that on the compression meridian (default value is 2/3)

The default parameters of the yield function were used in the analyses.
4.3.2.1.2. Steel Material

A multi-linear plastic material stress-strain relationship was assigned to the steel materials based on the uniaxial tensile test results. Each uniaxial test result was converted to the stress-plastic strain curves using Eq. 4-5.

For the reinforcing bars and other 1-D components the measured stress-strain curves were used, but for the solid elements (3-D objects) the “nominal stress-strain” curve $(\sigma_{\text{nom}} - \varepsilon_{\text{nom}})$ was transformed to “true stress-logarithmic plastic strain” relation $(\sigma_{\text{true}} - \varepsilon_{\text{in}})$ by using Eqs. 4-6 and 4-7:

$$
\sigma_{\text{true}} = \sigma_{\text{nom}}(1 + \varepsilon_{\text{nom}}) \quad (4-6)
$$

$$
\tilde{\varepsilon}_{\text{in}}^p = \ln(1 + \varepsilon_{\text{nom}}) - \frac{\sigma_{\text{true}}}{E} \quad (4-7)
$$

4.3.2.2. Element Types

Three dimensional linear brick elements with reduced integration (C3D8R) were chosen to model all the solid parts. The element is shown in Fig. 4-14. The reduced integration formulation significantly reduced the computational demand without compromising the accuracy of the results. To model longitudinal bars and spirals, embedded three-node quadratic beam elements (B32) were utilized. The beam element allows the model to captures bending in addition to axial deformations. This capability is critical if the dowel action of the reinforcing bars participates in load transfer mechanism.
A brief mesh sensitivity analyses demonstrated that a finer mesh is necessary adjacent to the pipe, as expected. The tension model for the CDP concrete material depends on the third root of volume of the element (the characteristic length of element), thus, flat or narrow concrete elements were avoided in critical regions.

4.3.2.3. Contact Between Surfaces

Interaction between the surfaces of the steel pipe, exterior can, and concrete was modeled by general contact algorithm. General contact uses a sophisticated tracking algorithm to ensure that proper contact conditions are enforced efficiently. The surfaces that were initially in contact such as the outer surface of the pipe and concrete, and the surfaces that could potentially come into contact such the top surface of the column and the soffit were defined as contact surfaces.

“Hard” contact was used for the normal interaction of the contact surfaces. In this algorithm, when the surfaces are in contact, any contact pressure can be transmitted between them. The surfaces separate if contact pressure reduces to zero, but are able to come into contact again if the clearance between them reduces to zero. This contact relation is approximated by stiff linear behavior. The stiffness is adjusted automatically to minimize penetration without adversely affecting the analysis time.

The tangential interaction of contact surfaces were specified with “Friction” model (Coulomb friction). The basic concept of Coulomb friction model is to relate the maximum allowable friction (shear) stress across an interface to the contact pressure between the adjacent bodies. The Coulomb friction model defines the critical shear
stress, \( \tau_{\text{crit}} \), at which sliding of the surfaces starts, as a fraction of the contact pressure, \( p \), between surfaces \( (\tau_{\text{crit}} = \mu p) \). The coefficient, \( \mu \), was assumed to be equal to 0.425 [Baltay, and Gjelsvik, 1990; Rabbat, and Russell, 1985].

4.3.2.4. Analysis Method

With respect to the solver methods of ABAQUS package, “Explicit Solver” performs better than the “Standard Solvers” when it comes to the high curvature contact surfaces and softening materials such as concrete. The loading was displacement control. Applying the displacements with a constant velocity introduces noise in the response because of the initial momentum (impact). Thus, the lateral load was applied by defining a gradually increasing velocity boundary condition.

4.3.2.5. Sensitivity Analysis

The concrete material properties were the least known parameters in the FE model. To establish sensitivity of the results to changes in each parameter, a comprehensive sensitivity analyses was conducted on the PS2P-A/B (Sec. 2.2) model. The parameters of concern were: concrete dilation angle (\( \psi \)), \( K_c \) factor, \( \sigma_{ho} / \sigma_{co} \) ratio, friction coefficient, contact algorithm, fracture energy of the concrete material, and tensile strength of concrete.

The performance of the model in pull and push was of concern during the calibration of the FE models. Figs. 4-15 through 4-21 show the results of the sensitivity analysis. The
values of the parameters that were selected for analyses are identified in bold in the legends of the figures.

Parameters such as $K_c$, $\sigma_{bo}/\sigma_{co}$, and dilation angle, that define the plasticity of concrete and effect of tri-axial stress field on the concrete behavior significantly affected the push results. As expected, parameters that are related to fracture and cracking of the concrete mostly impacted on the pull loading results. Friction coefficient demonstrated some impact on the push results (Figs. 4-18). It is because larger friction provided better confinement for the concrete next to the pipe and did not allow for bulging towards the free surface, which accordingly increased the bearing strength of the concrete due to a better confinement. The difference between the results of different contact solver method (Fig. 4-19) was significant, and the Kinematic Solver Method showed considerably better agreement with the test results.

4.3.2.6. Analytical Outputs

FE analyses generate massive amount of data, including: stress fields, strain fields, support reactions, contact stresses, energy balances, and etc. The most relevant outputs are the ones that can briefly define state of damages, including yielding, plastic deformation, and cracking. In this research, the main focus was on the load-displacement responses.

In addition to the load-displacement curves, some of the stress and strain distributions were studied to identify plastic deformations and fractures. To locate the yielding of steel components, Von Mises stresses were plotted after each analysis. Von Mises stress is a
yield criterion for ductile materials such as mild steel which is defined in Eq. 4-8. If the Von Mises stress exceeds the uniaxial yield stress material yields.

$$\sigma_{Von Mises} = \sqrt{\frac{3}{2} s_{ij} s_{ij}}$$  \hspace{1cm} (4-8)

Where $s_{ij} = \sigma_{ij} - \frac{1}{3} \delta_{ij} \sigma_{kk}$ is the stress tensor deviator.

The equivalent plastic strain, PEEQ, was used as a quantitative measure for extent of the plastic deformations. This value is a scalar quantity that is defined by Eq. 4-9.

$$PEEQ = \sqrt{\frac{2}{3} \varepsilon_{ij}^{p} \varepsilon_{ij}^{p}}$$  \hspace{1cm} (4-9)

Another output parameter that was monitored in the analyses was PEEQT. This scalar quantity is called equivalent maximal plastic tensile strain. This value is a measure of the crack width, because the tensile behavior was defined in terms of stress-displacement. This parameter shows pattern and extent of cracking in brittle material, including concrete.

4.3.3.  FE Model for Push-Off Specimens

Because of planar symmetry, only one half of the geometry of the specimen was modeled. Effect of symmetry was introduced to the model by restraining the out of plane displacements along with rotations about the perpendicular axis in the plane of symmetry. The models comprised the following components: concrete body, concrete inside the pipe, steel pipe, reinforcing bars, and spiral/s around the pipe. Overall, approximately
17000 elements were used in each model and the running time was approximately four hours.

The parameters used for the concrete material are listed in Table 4-2 for the main body of the specimens and infilled pipe. Steel was modeled based on the stress-strain curves that were presented in Chapter 2. Figure 4-22 shows the details of PS1P specimens. In this figure, the concrete elements in the top half of the model are excluded to show the details of the inner reinforcement. The load was applied through the rigid plate that is shown on the left side of the figure.

Figures 4-23 to 4-25 compare the calculated and measured load-displacement curves. It can be seen that there was a close acceptable correlation between the results. Overall, the analytical model performed well in terms of estimating elastic stiffness, yield regions, post-yielding stiffness, and ultimate capacity of the push-off specimens.

The stress field obtained from the FE analysis of PS1P and PS2P are presented in Figs. 4-26 and 4-35. The results of the PS3P are very similar to the ones of PS1P and are not presented in this report. Figures 4-26 and 4-31 demonstrate deformed shape of the specimen. The model was not setup to remove the fully damaged elements during the analysis. The bulged area in the front of the pipe in Fig. 4-26 was fully damaged and could not carry any more stresses. Figure 4-27 shows the pattern of the tensile damage when specimens were pushed. The value of PEEQT and its pattern were similar in both specimens, which show that inner spiral did not play a significant role in controlling tensile damages under push loading.
Dark area in Fig. 4-28 shows the extent of plastic deformations in the specimens. This figure illustrates that the bearing failure of concrete against the pipe is a localized damage. Comparison of Figs. 4-28a and 4-28b reveals that the plastic region was larger in PS2P because of absence of the inner spiral. The normal contact stresses are shown in Fig. 4-29. By increasing the plastic deformations the surface concrete lost its bearing capacity and maximum contact stress (which is also a measure of the bearing stresses) shifted to larger depth. In PS1P, the maximum contact stresses occurred in lower depth because the inner spiral limited the concrete damage at lower depths. Figure 4-30 shows the Von Mises stresses on the pipe. In both cases, the pipe fully yielded in flexure and large plastic shear deformations also occurred at middle of the pipe.

Figure 4-32 demonstrates an important fact. The extent of tensile damage in the specimen that included inner spiral is much less when the specimen was pulled. As Fig. 4-32b shows, the tensile cracks could extend to the surface of the specimen in PS2P (these cracks were detected during the experiment, as well); but, in PS1P the inner spiral prevented the spread of tensile damage. No significant flexural or shear yielding was observed under pull loading as Fig. 4-35 indicates.

4.3.4. FE Model for PF-1

4.3.4.1. Details of PF-1

PF-1 was a 1:3.5 scaled hinged column of standard Deign-A that was build and tested under quasi-static loading at the Large Scale Structure Lab at University of Nevada, Reno [Doyle and Saiidi, 2008]. The geometry and reinforcing detail of PF-1 are illustrated in
Fig. 4-36. The column diameter was 22 in (559 mm). On the test day, concrete compressive strength for the column and the loading head were 7.1 ksi (48.95 MPa) and 4.6 ksi (31.44 MPa), respectively. The measured yield and tensile strength of longitudinal bars were 71 ksi (490 MPa), and 110 ksi (760 MPa), respectively. The spiral around the pipe had an average yield and tensile strength of 65 ksi (447.5 MPa) and 75.6 ksi (521.3 MPa), respectively. The 3-1/2 x-Strong steel pipe met the A53 Gr. B steel specification with the yields stress of 45.5 ksi (313.7 MPa) and tensile strength of 68.2 ksi (470.2 MPa).

The loading head represented the superstructure. Because in multi-column bents, the superstructure remains essentially horizontal with no rotation under lateral loads, two actuators were used in the test to apply the lateral load on the loading head and restrain it from rotating (Fig. 4-37).

Applied axial load on the column was 100 kips (445 kN). Friction coefficient was expected to decrease under cyclic loading. Therefore, only at low amplitudes (prior to yielding of column) full cyclic loads causing equal displacements in both directions were applied. The ultimate load was monotonically applied after the column yielded to avoid unrealistic reduction of friction. Lateral loading of the column was displacement controlled.

Figures 4-38a to 4-38c show the measured force-deformations in terms of hinge (1) slip, (2) loading head displacement, and (3) displacement at top of the column immediately below the hinge. Up to a certain load, entire lateral load at the hinge was resisted by
friction and slippage at the interface was insignificant. Under approximately 80 kips (356 kN), the friction was overcome, the head slipped relative to the column, and the side gap between the pipe and the can was closed. The sudden drop in the load-displacement curves of Figs. 4-38a and 4-38b signify the release of friction. After the steel can came in contact with the pipe, the lateral load increased until the pipe-pin started to yield. Extensive flexural cracks and some shear cracks were developed on the concrete column as cyclic loads increased. The test was stopped at a column drift ratio of 10%. The maximum slip between the loading head and the column was 1.3 in (33 mm).

4.3.4.2. FE Model for PF-1

Modeling the entire setup of PF-1 with solid element was computationally too demanding and not necessary. Only the local behavior the pipe-pin was of the interest in these analyses. Therefore, only top 30 in (762 mm) of the column was modeled using 3-D brick elements and the rest of the column was modeled by means of a frame element (Fig. 4-39a).

Performance of the pipe-pin is essentially affected by the magnitude of rotation at the top of the column when the column bends under lateral loading. The source of this rotation is mainly nonlinear rotation that occurs at plastic hinge at the base of the column. A lumped plastic hinge at the base was utilized to account for flexural yielding of the column.

Initially the properties of the column plastic hinge were defined based on moment-curvature analysis of the column section. However, because of approximations in
estimating the bond-slip rotation at the base and shear deformation of the column, the model underestimated the displacement and rotation at top of the column. Because the focus of the FE modeling of PF-1 was to study the pipe-pin connection, the experimental data was used to find the properties of the column plastic hinge in PF-1. To obtain the plastic deformation at the top of the column, the theoretical elastic and shear deformations were subtracted from the total measured column displacement.

PF-1 was analyzed under monotonic loading. Figure 4-40 compares the calculated and measured load-slip relationship at the pipe-pin hinge. It is clear that the model led to a very close estimate of the ultimate capacity. The only parameter that was tuned in the FE model was friction coefficient between concrete surfaces. The friction coefficient was selected 0.85 to capture the correct release force of 80 kip (356 kN).

Figure 4-41, and 4-42 show the pattern of the cracks around the pipe and Von Misses stresses on the pipe, respectively. Von Misses stresses in Fig. 4-42 confirm the extensive flexural yielding of the steel pipe at a depth of approximately 1.5 times diameter of the pipe. Also, the pipe yielded locally in shear at the concrete surface (the dark region on the pipe-pin in hinge throat area in Fig. 4-42). Figure 4-43 shows the deformed shape. It confirms that top edge of the pipe touched the can when the pipe bent due to yielding. The slight increase in the force at 1.2-in (30.5-mm) slip in Fig. 4-40 was the result of the moment generated at the pipe.
4.4. OpenSees Model for the Two-Column Bent

4.4.1. Introduction

OpenSees is an open-source software for nonlinear seismic analysis of structures. This program includes several material models, element types, and analysis methods.

Modeling the two-column pier specimen using this program served three purposes. First, the pre-test analysis results were used to design the shake table model and to select the ground motion such that the main purpose of the proof test is satisfied without exceeding the physical limitations of the shake table facilities as explained in Chapter 2. The second reason for modeling the bent in OpenSees was to develop and verify a macro model for pipe-pin hinges that can be used in real bridge models. And the third reason was to develop a reliable analytical model to be used for parametric studies.

4.4.2. Components of the Model

The analytical model is shown in Fig. 4-44. The model was a two-dimensional representation of the test pier for in-plane analysis.

The cantilever columns were modeled using fiber sections. Section of the RC column included: confined concrete fibers in the core, steel fibers as longitudinal bars, and unconfined concrete fibers for the cover. The FRP tube column section was composed of: FRP confined concrete fibers for the encased concrete, steel fibers for the longitudinal bars, and FRP fibers to represent the longitudinal behavior of the FRP shell.
The cap beam was modeled as a linear beam element. The two end nodes of the beam were slaved to the column top nodes. The horizontal DOFs of the ends of the bent element were connected to the columns through two nonlinear horizontal truss elements to model the horizontal behavior of the pipe-pin hinges. The purpose of the truss elements was to model the effect of slippage between the cap beam and the columns. No rotational springs was defined at the pipe-pin hinges. The moment was released at the top of the columns assuming moment-free hinges.

In the pre-test model the base of the columns were fixed, but in the post-test analysis model a nonlinear rotational spring was assumed at the base to take the bond-slip rotations into account.

94.7% of the total mass was modeled at the level that the mass rig link and was connected to the cap beam. 2.5% of the total mass was introduced to the two end nodes of the cap beam, and the remaining mass was distributed equally between the top nodes of the columns. The mass was distributed to better represent the actual mass distribution and avoid numerical convergence problems.

4.4.3. Material Models

4.4.3.1. Concrete

The “uniaxialMaterial Concrete01” was used to model the unconfined and confined concrete fibers. This is a uniaxial concrete model based on Kent, et al. with degraded linear unloading/reloading stiffness according to the study by Karsan and Jirsa and no tensile strength [OpenSees Manual, 2005]. The tensile strength of the concrete was
neglected everywhere to account for the possible existing thermal and shrinkage cracks. The Mander’s [1988] model was used to determine the properties of the confined core in RC column. The Saiidi’s [2005] confinement model was used for the FRP encased concrete in the FRP tube column. In this model the module of elasticity of the FRP tube in the hoop direction was assumed to be 1850 ksi (12755 MPa) and the rupture stress of the fiber assumed to be 34 ksi (234 MPa) based on the material properties that were presented in Chapter 2.

Modeling parameters of the concrete material are presented in Table 4-3 for pre-test and post-test analytical models. Unconfined concreted strength for the pre-test analysis was 5.5 ksi (37.92 MPa). The concrete strength used in the post-test analysis was 8.58 ksi (59.16 MPa) due to the high rate of loading, as will be explained at the end of this chapter.

4.4.3.2. Steel

The “uniaxialMaterial Steel02” material was used for steel in the pre-test analysis with yield strength of 64 ksi (441.28 MPa) and strain hardening ratio of 0.02. For the post-test analysis it was noticed that Steel02 material deviates from the stress-strain curve of steel in large strains (Fig. 4-45). Therefore, the post-test analysis was carried out twice once with Steel02 material model and once using “ReinforcingSteel” material model. For the post-test analytical model, the yield strength and strain at start of strain hardening ratio were assumed to be 74.97 ksi (516.9 MPa) and 0.03, respectively, based on the material test results as presented in Chapter 2. The same yield stress was used for the
ReinforcingSteel material. Other parameters were: 1360 ksi (9377.2 MPa) for the initial hardening slope, 123 ksi (848 MPa) for the ultimate strength, and 0.15 for ultimate strain. Figures 4-45 and 4-46 show the stress-strain hysteresis curves obtained from the two steel material models compared to the measured stress-strain relationship after it was modified for strain rate effects. These figures show that the hysteresis curves have larger area in ReinforcingSteel material compared to area for the Steel02 material model. A similar effect was also seen in the force-displacement curves as will be explained in subsequent sections.

4.4.3.3. FRP Tube

Glass fibers in the FRP tube were aligned at ±55° to provide strength in hoop as well as longitudinal directions. Longitudinal behavior of the FRP tube was defined using a model that was proposed by Mirmiran et al. [2005]. This model assumes a tri-linear “uniaxialMaterial Hysteretic” material to define the longitudinal behavior of the FRP tube. Figure 4-47 shows the stress-strain hysteresis curve associated with this material. Required points in the model are shown on the graph. The parameters: pinching factor for strain, pinching factor for stress, damage due to ductility, damage due to energy, and degrading factor of unloading stiffness were taken as 1, 1, 0, 0, and 0.3, respectively [Mirmiran et al., 2005].

4.4.4. Pipe-Pin Hinge Macro Model

There was no appropriate element model in OpenSees to represent lateral behavior of the pipe-pin hinges. A compound element was developed in the present study that was
capable to duplicate force-slip behavior of the pipe-pin hinges. The parameters in this element are the initial stiffness of the hinge, friction release force, and yield force and displacement of the hinge.

The proposed element comprises three subcomponents. Figure 4-48 presents an overview of the element. Two Steel02 material model and one gap model were used in series and parallel configuration to generate the required response. NL Spring-1 in this figure defines the yielding and ultimate capacity of the connection. The parallel combination of the gap element and the NL Spring-2 element duplicates effect friction in the model. When the pipe-pin model is loaded, the load is transferred through a series of NL Springs until NL Spring-2 reaches the “yield” load (friction threshold). After NL Spring-2 yielded, the gap becomes free to move until it locks (associated with the gap closure between pipe and can). A large stiffness is assigned to the gap, and as a result, the friction element of the model (NL Spring-2) does not deform any further. By increasing the load, the NL Spring-1 deforms linearly until it reaches the yield force. Yielding of NL Spring-1 takes the yielding of the connection into account. This model is capable to allow for Bauschinger effect that was observed in cyclic analysis of the connection as is explained in Chapter 5.

This compound macro model was assigned to a uniaxial truss element with unit length that was located between the end of the cap beam and top of the column. Figure 4-49 shows the model that was used in pre-test analysis. The model was modified based on the measured steel and concrete properties and then was used in the post-test analyses. Behavior of this model is presented in Fig. 4-50. The experimental results showed that
the stiffness of the hinge model when the pipe comes in contact with the can is less than what was initially assumed because of the rotation that occurs at top of the column.

4.4.5. Pre-Test Analysis

Results of the OpenSees model that was used to design the two-column pier model are presented in this section. To be able to compare the results with experiments, the pier was analyzed using the achieved acceleration histories of the shake table. Figures 4-51, 4-52, and 4-53 compare the load-displacement hysteresis curves obtained from analysis and experiment for the bent, RC column, and FRP tube column, respectively. These curves demonstrate that the pre-test analysis model under predicted the loads and over predicted the displacements. The overall shapes of the hysteresis curves were compatible as far as the pinching and unloading slopes are concerned.

Figures 4-54 and 4-55 compare the pre-test analysis and measured results for the pipe-pin hinges. These plots show that the calculated initial friction release force was higher that the measured one. Also the second slope of the curve when the pipe touched the can, was overestimated significantly.

4.4.6. Post-Test Analysis

4.4.6.1. Strain Rate Effect

As noted in the previous section, the measured lateral forces were larger than the ones predicted by pre-test analysis. Even after using the actual test day material properties, there was still a considerable difference between the results that needed to be addressed.
by other means. After reviewing the achieved strain data during the test, it was noticed that the strain rates were very high as explained in Chapter 3. It is well understood that the material strength increases by increasing the rate of loading. There are several formulations proposed for the strain rate effect to modify the properties of steel and concrete material. Saiidi, et al. [2007] proposed Eq. 4-10 for steel. In this equation the average of the strain rate between the yield strain and half of the yield strain is used as the strain rate value. However, it was noted in the measured strain data that variation of strain rate was not significant in the range of interest. Therefore, the maximum strain rate recorded in this range was used in this equation to find the strain rate factor.

\[
SRF = \left( \frac{\dot{\varepsilon}_{\text{average}}}{100} \right)^{0.0129}
\]

\((4-10)\)

**SRF**: Strain rate factor

\(\dot{\varepsilon}_{\text{average}}\): Average strain rate when the strain in steel is between \(0.5\varepsilon_y\) and \(\varepsilon_y\)

Applying a single factor to the entire stress-strain curve of steel may be not accurate, because strain rates increase when the maximum strains increase in progressive runs. Figure 4-56 demonstrates this trend. This figure shows the correlation between the maximum strain rates in the range of concern and the maximum tensile strain recorded at each run. Each curve is the average of four strain gauges that are located at the plastic hinge of area of the columns. Based on these curves the strain rate factor was presented in terms of the maximum strain using a linear regression according to Eq. 4-11. This
equation is only valid for this experiment and is generally a function of the ground motion.

\[
SRF = 1.085 + 6.5 \times 10^{-7} \varepsilon_{\text{max}}
\]  

(4-11)

Figure 4-57 compares the original and modified stress-strain curves for the longitudinal bars. The yield and ultimate strength showed 8.6% and 14.4% increase, respectively, after applying the strain rate effects.

The concrete strength was also modified to account for the strain rate effect. Equation 4-12 was used based on a study done by Kulkarni et al. [1998]. For the concrete, the average strain rate when the strain in the longitudinal bar was approximately 0.006 (the crushing strain of concrete) was 92500 microstrains.

\[
SRF = 0.0222 \ln(\dot{\varepsilon}) + 0.9973
\]  

(4-12)

The strain rate factor for concrete was approximately 1.25. This factor was multiplied on the test day strength of 6.87 ksi (47.4 MPa) and resulted to 8.58 ksi (59.2 MPa). This value was used to determine the properties of the steel and FRP confined concrete cores.

4.4.6.2. Bond-Slip Model

The bond slip effects were not included in the pre-test analytical model. Bond-slip rotation is the result of yield penetration of the longitudinal bars into the footing. The bond-slip effect can be modeled with a lumped nonlinear rotational spring at the bottom of the columns.
Saiidi and Wehbe [1999] developed a method to calculate the bond-slip rotations associated to cracking, yielding, and ultimate capacities of the RC columns. This method was utilized to find the properties of the rotational spring at the base of the RC column. Figure 4-58 compare the calculated and measure bond-slip rotations. This figure demonstrates a good correlation between the results.

For the FRP tube column, there was no established method that addresses the bond-slip properties of the embedded tubes. Due to lack of a theoretical method, the experimental data were used to model the bond-slip spring at the base of the FRP tube column. Developing a new bond-slip rotation model for FRP tube columns was beyond the scope of the present study. Figure 4-59 compares the measured bond-slip rotation curves with the bilinear model used in analyses.

4.4.6.3. Axial-Load History

It was mentioned in Chapter 3 that the axial load of the columns underwent large variation during the test. The initial axial load was reduced prior to the last three runs to avoid excessive axial load peaks during the large-amplitude shake table tests. To closely represent the axial load variation, the axial load was modeled as a variable that was a function of time. The axial load time history was obtained from the readings of the load cells during the tests. Figure 4-60 and 4-61 compare the applied axial load history and that used in the model during the last run. The insignificant difference between input and achieved axial load histories is attributed to the dynamic response of the model in vertical direction.
4.4.6.4. Results

Figures 4-62 to 4-67 present the load-displacement results for the bent, RC column, and FRP tube column. Both steel elements closely simulated the experimental results in terms of forces and displacements. Steel02 model was more successful in simulating the unloading slopes. The hysteresis curves obtained from the ReinforcingSteel material had larger areas comparing to what was obtained from the test. Figure 4-68 to 4-81 show the displacement histories for all the runs. The analytical model overestimated the displacement during the low-amplitude runs 1 through 4, but led to good correlation during stronger motions. The results were up to 100% off in low amplitude motions, but the error was reduced to less than 20% under large motions. The large difference during the initial runs was due very large damping that the model experienced in early runs as was discussed Chapter 3.

The base shear histories for the columns are plotted in Fig. 4-82 and 4-83. At low amplitudes, the RC column took a larger share of the total load, but at higher amplitudes the FRP tube column had the larger share of the load because RC column yielded and stopped attracting larger forces whereas the extent of nonlinearity in the FRP tube column was limited because of linear behavior of FRP material.

Figure 4-84 to 4-87 compare the analytical and experimental slippage at the hinges. The proposed element for the pipe was able to duplicate the load-slip behavior of the pipe-pins very well. The maximum measured and calculated slips were approximately the same.
Figs. 4-88 to 4-91 show the rotation histories at top of the columns. The analytical and experimental results were similar in the positive direction, but the correlation in the negative rotation zones was not close. This was because the Novotechnik transducers that measured the rotations at top of the columns ran out of stroke.

The calculated natural period of the structure before and after the analysis was 0.264 second and 0.649 second. These periods are comparable to the measured periods of 0.245 second and 0.714 second.
5. PARAMETRIC STUDIES OF COLUMNS WITH PIPE-PIN CONNECTIONS

5.1. Introduction

To develop a general design method for pipe-pin hinges, the effect of all components of the hinge on capacity and performance should be known. Parameters such as geometry, material properties, and reinforcement could influence the lateral load capacity of pipe-pins. Due to time and cost limitations, it is not feasible to study the role of each variable experimentally, and analytical studies are necessary. To understand and quantify the importance of each parameter, an extensive parametric study was performed using elaborate finite element (FE) modeling.

The FE model was first calibrated and checked against the experimental data that was generated as part of this project. The modeling details, calibration process, and performance of the FE model were explained in Chapter 4. The basic prototype model that was used in the study is first described. Then the parameters and the range used for each are explained and justified. The FE results are presented subsequently and the trends of the sensitivity of the results are discussed.

5.2. Prototype Model

Caltrans pipe-pin “Design A” standard detail was selected as the prototype detail. This standard pipe-pin design has been incorporated in different column sizes as presented in
Table 2-5. For the purpose of parametric study, the pipe-pin detail was assumed to be incorporated in a column with diameter of 72 in (1829 mm), which is the average diameter of the columns using pipe-pins in the San Francisco Bay Bridge approach spans. Specifications of “Design A” pipe-pin were described in Chapter 1 in this report. The studs were not included in the model because it is unlikely that they will be used in future bridges based on the results of the current study, which found studs are not needed.

Modeling the entire length of the column with a fine mesh of solid elements is computationally demanding and unnecessary for the current study. This is because the main reason for FE modeling was to capture the pipe-pin behavior and to focus on local failures; hence, only the top 108 in (2743 mm) upper part of the column along with a small portion of the cap beam were modeled using solid elements (Fig. 5-1a, b) to reduce the analysis time. The analysis, however, had to include the effect of column rotation on the pipe-pin response. In Chapter 4 the effect of column flexibility in PF-1 was represented using a beam-column 2D frame element. However, that method was not used in the study presented in this chapter because to capture the load-deformation response of the pipe-pin, the cap beam had to be pushed to a very large displacement due to column flexibility. For instance to generate a 3-in (76.2 mm) relative slip at the hinge, the column cap needed to be pushed by approximately 15 in (381 mm), which required substantial amount of analysis time. Therefore the flexibility of the lower part of the column was modeled by a rotational spring. Figure 5-2 illustrates the model. In this figure, $L_c$ and $L_h$ are the entire length of the column and length of the top portion of the column which was modeled using a fine mesh, respectively. The only error associated
with this idealization was underestimation of the P-Delta effect, which was not of a concern in the parametric study.

The stiffness of the rotational spring was obtained using the following calculations:

\[
\theta = \frac{H(L_c - L_h)^2}{2EI} + \frac{HL_h(L_c - L_h)}{EI} \quad (5-1)
\]

On the other hand:

\[
\theta = \frac{M}{K_\theta} = \frac{HL_h}{K_\theta} \quad (5-2)
\]

Therefore, the equivalent rotational stiffness of the spring is:

\[
K_\theta = \frac{2EIL_h}{(L_c - L_h)^2} + \frac{EI}{(L_c - L_h)} \quad (5-3)
\]

\( \theta \) = Rotation at the location below which the column is not modeled

\( H \) = Lateral load

\( EI \) = Flexural stiffness of the column

A yielding column would control the load and would not allow for the lateral load capacity of the pipe-pin to be determined. Therefore the lower part of the column was treated as a linear member.

The cap beam is typically stiff and is highly confined. Therefore, the outer surfaces of the modeled portion of the cap beam were modeled by a rigid surface with only the in-plane translational degrees of freedom allowed as shown in Fig. 5-1. Only one half of the system was modeled due to geometric symmetry.
The prototype model has \#8@5 in (127 mm) as column spiral and \#5@3 in (76 mm) as inner spiral around the pipe and the exterior can. The pipe diameter is 14 in (355.6 mm) with a thickness of 1.25 in (31.7 mm). The embedded and protruded lengths of the pipe are 60 in (1524 mm) and 24 in (610 mm), respectively. The exterior can thickness and the gap between pipe and exterior can are both 0.5 in (12.7 mm). The height of the hinge throat is 1 in (25 mm).

Strain hardening was neglected in all the steel materials in order to be able to use the yield capacity of the steels in the design method. Yield strength of the steel pipe/can, and reinforcing bars were assumed to be 45 ksi (275.7 MPa) and 65 ksi (398.3 MPa), respectively. The strength of the concrete was assumed to be 4.5 ksi (27.6 MPa) for the column and the bent cap. The aspect ratio of the column was assumed to be 4.0 with 2% longitudinal reinforcement. The axial load on the base model was 1100 kips (4893 kN). The friction coefficient was assumed to be 0.7 for the base prototype model which is slightly conservative compared to the corresponding value of 0.8 obtained from PF-1 test.

Dynamic analysis at a very slow rate was used because the Explicit solver (ABAQUS solver which works in time domain) was more powerful as explained in Chapter 4. The loading of the model consisted of two steps: (1) gradually applying the full axial load in the force control mode, and (2) applying the lateral load by imposing increasing lateral displacement to the cap beam in a displacement control mode. The displacement had to be applied slowly and at small increments. Otherwise, the model would vibrate as a result of the impact loading and noise would be introduced in the response. To solve this problem, displacement was applied by defining a linearly increasing velocity which
started from zero and remained constant after reaching 0.1 in/sec (2.54 mm/sec) until 5 in (127 mm) displacement was achieved. Each run took approximately 4 hours using a PC with a Quad core CPU of 2400 MHz.

5.3. Parameters and the Results

5.3.1. Introduction

The prototype model was used to study the influence of different design parameters on the performance and capacity of pipe-pin hinges. Table 5-1 lists the parameters and their ranges. The parameters were changed one at a time, but for some of the parameters that were expected to interact with each other, a combination of interacting parameters was used for the parametric studied. For example, combinations of different friction coefficients with different axial load levels were analyzed.

As previously stated, the diameter of the bearing area varies depending on the expected rotation at the top of the column. For longer columns (e.g. the upper deck of approach ramps of the San Francisco Bay Bridge) the bearing area (hinge throat) diameter is 1 ft (0.305 m) smaller than the column diameter. For shorter columns (e.g. the lower deck of approach ramps of San Francisco Bay Bridge) the bearing area diameter is 3 ft (0.914 m) and is constant. In the parametric studies, the cases in which the diameter of bearing area could have significant effect on the behavior were repeated for two bearing diameters of 36 in (914.4 mm), case (1), and 60 in (1524 mm), case (2). The shaded cells in Table 5-1 indicate the parameters for which two different bearing areas were used.
The same as the models that were presented in Chapter 4, the results recorded during the analysis included: Von Misses stress to capture yielding of the steel parts, tensile damage measure (PEEQT) to study extent and pattern of cracks in concrete material, and equivalent inelastic strains (PEEQ) to capture failure of concrete in compression. The intensity of deformations was monitored at degrees of freedom defined on Fig. 5-3. The forces that were recorded were moment at the rotational spring, lateral force at the support, and the lateral force at the top middle point of the cap beam, which was the point on which lateral displacement was imposed.

For each parameter, two sets of curves are presented in this report. The first shows the hinge slip versus lateral load. Hinge slip was obtained by subtracting the horizontal displacement of the top right corner of the column from that of the cap beam middle node. The second set of curves shows the pipe rotation against the lateral force. The pipe rotation was calculated by dividing the difference of the left and right vertical movements of the top corners of the pipe by the pipe diameter. A set of three stress/strain plots also is presented for each case. These stress gradients show the Von Misses stresses and crack widths in the model.

5.3.2. **Friction Coefficient and Axial Load Index**

A part of the lateral load in pipe-pin hinges is transferred through friction at the concrete contact area. There is commonly a considerable scatter in friction forces because of dissimilarities in micro-scale geometry of the concrete surfaces, and also uncertainty of the value of existing axial load. The design engineer should be cautious about relying on
friction as a resisting force. Different combinations of friction coefficients and axial loads were examined to study the sensitivity of the pipe-pin capacity to these two parameters. Four friction coefficients of 0, 0.3, 0.7, and 1.0 were studied. For each friction coefficient except 0 and 0.7, analysis was repeated for two different axial load indices (ALI) \((ALI = P / A_g f'_c)\) of 0.06, and 0.10 representing low and high levels of axial load on the column, respectively. \(A_g\) and \(f'_c\) are the gross cross sectional area and the concrete compressive strength of the column. These axial load indices correspond to 1100 kips (4893 kN) and 1850 kips (8229 kN) of axial load on the column, respectively. For the friction coefficient of 0.7, ALI=0.13 (2400 kip, 10675 kN) was also examined besides 0.06 and 0.1 to see the effect of extra high axial loads. For the zero friction coefficient, analysis was also repeated with no axial load to find the lower limit of hinge capacity.

In Fig. 5-4 the results for the zero friction coefficient are presented. The load initially remained zero until the vertical gap closed and the exterior can touched the pipe-pin. With no axial load, the load-deformation curves are similar for cases (1) and (2) (cases (1) and case (2) are associated with the small and large bearing areas, respectively). This observation is intuitive, because the bearing size shows its significance only when axial load is present.

On the other hand, the capacity of pipe-pin hinge improved when the axial load on the column increased, even when the friction coefficient was zero. The positive effect of axial load on hinge capacity increased with axial load. Figure 5-4 shows that the capacity
of pipe-pins with smaller bearing area for ALI=0, 0.06, and 0.1 is 914 kip (4065 kN), 1300 kip (5782 kN), and 1370 kip (6094 kN), respectively. This is because the shear capacity of concrete is enhanced by an increase in normal stresses. As will be explained at the end of this chapter, the dominant mode of failure of pipe-pin hinges is associated with shear cracking of the concrete adjacent to the steel pipe. A larger bearing area reduces the impact of the axial load. This is because, with a larger bearing area, the normal compressive stress on concrete is smaller for the same axial load; therefore, the increase in capacity is lower.

Figures 5-5 to 5-7 show the load-displacement response of pipe-pin hinges for friction coefficients of 0.3, 0.7, and 1.0, respectively. The lateral force was initially resisted by friction. The point at which the maximum friction resistance is reached is indicated by the general drop in the load in the curves due to slippage, which occurred at approximately 0.25 in (6.35 mm). With larger friction coefficients, the maximum friction force increased with an increase in the bearing area. This occurs because the resultant stress at the edge of the hinge throat due to the combined action of friction force and vertical stress is lower when bearing area is larger. As a result, a larger capacity is realized. The extent of Von Misses stresses at the edge of the hinge throat in Figs. 5-52 and 5-55 as pointed by solid arrows, confirms this hypothesis.

Increasing the friction coefficient also increased the ultimate capacity. For example, the lateral capacities for the larger bearing area when friction coefficient is 0.7 were 1523 kip (6783 kN), 1831 kip (8144 kN), and 1950 kip (8673 kN) for axial load indexes of 0.06, 0.1, 0.13, respectively. In the presence of axial load, friction contributed to the hinge
capacity but not in a linear proportion with the friction force (i.e. axial load multiplied by the friction coefficient). The ultimate capacities increased for larger bearing area. This is attributed to the larger diagonal tension failure plane associated with the larger bearing area. The dashed lines in Figs. 5-39 and 5-42 show the approximate size of the shear planes for the two bearing area diameters when friction coefficient is 0.7 and $\text{ALI}=1.0$.

The capacity did not significantly improve when friction coefficient increased from 0.7 to 1.0 because, as will be explained in subsequent sections, shear cracking controlled the capacity.

The aforementioned results prove that the axial load plays two distinct roles: 1) increasing the friction force, and 2) enhancing the intrinsic capacity of the pipe-pin; in other words, even when the friction coefficient is zero, axial load increases the capacity.

5.3.3. Column Spiral

PF-1 test demonstrated that failure of the pipe-pin is associated with diagonal cracking of the concrete surrounding the pipe that extends through the column spiral. Therefore, the amount of column spiral reinforcement can be an important parameter. A practical range of column spiral design was assumed to determine the influence of this parameter on the pipe-pin shear capacity: #8@10 in (254 mm), #8@5 in (127 mm), and 2-#8@5 in (127 mm). The volumetric steel ratio for these cases is 0.47%, 0.93%, and 1.86%, respectively.

Figure 5-62 shows the load-deformation curves and Figs. 5-63 through 5-72 show the crack patterns, and Von Mises stresses associated to this parameter. Figure 5-62
demonstrates that the lateral reinforcement had a significant effect on the lateral load capacity of pipe-pins. The load-displacement curves are very similar until the load reaches 1200 kips (5338 kN) as shown by an arrow. At this load diagonal cracking began. After this point, the higher the shear reinforcement, the higher the lateral load capacity. As Fig. 5-62 illustrates, the capacity decreased after the start of cracking load when the spiral amount was very low (i.e. #8@10 in, 254 mm). The load reaches 1520 kip (5338 kN) for #8@5 in (127 mm) and over 1900 kip (8451 kN) when spiral ratio was doubled. Comparing the area encased in the ovals in Figs. 5-67 and 5-73 demonstrate that the extent of cracking was significantly reduced when the volume of the spirals was increased.

This observation proves that the column spirals are necessary to help resist diagonal tension across the cracks, the same way that spirals resist shear in RC columns. Figure 5-62 shows that sufficient shear reinforcement must be provided by the designer at the pipe-pin hinge area to ensure ductile behavior. Extension of the column spiral may not be sufficient, and higher amount of spiral reinforcement might be necessary in the upper part of the column to provide the required capacity of the pipe-pin.

5.3.4. Inner Spiral

The inner spiral has a role similar to that of the column spiral in resisting the diagonal tension near the pipe-pin. Three different value of inner spiral reinforcement were selected to study its effect. In addition to the amount of reinforcement, the diameter of the inner spiral was also varied. The inner spiral was changed to #5@6 in (152 mm),
#5@3 in (76 mm), and #7@3 in (76 mm) each for two different cage diameters of 30 in (762 mm), and 42 in (1067 mm). The range of variation of this parameter corresponds to the #5@3 in (76 mm) with a cage diameter of 30 in (762 mm) in prototype detail. Figures 5-75 and 5-76 show the load-deformation curves. Figures 5-77 through 5-106 present the stress/strain contours.

Figure 5-75 and 5-76 show the influence of the inner spiral amount on the capacity for spiral diameter of 30 in (762 mm) and 42 in (1067 mm), respectively. It can be seen that increasing the amount of the inner spiral around the pipe slightly increased the capacity. The effect of the amount of the inner spiral is realized after diagonal cracking of concrete. The capacity increased by approximately 12% and 15% for the smaller and larger bearing area diameters, respectively; when inner spiral was changed from #5@6 in (152 mm) to #7@3 in (76 mm). It is evident that the capacity enhancement from inner spirals is less than that of the column spirals because the crack plane intersects a smaller number of inner spirals than the column spirals. Comparison of Fig. 5-75 and 5-76 shows that the capacities are quite similar for two hinge bearing diameters indicating that there is little interaction between the inner spiral and the bearing diameter.

For a similar pitch and bar diameter, the larger spiral diameter resulted in a slightly higher capacity because as the spiral diameter increases, a larger number of bars intersect the diagonal crack. Besides, anchorage of the spiral bars in concrete improved by increasing the cage diameter. Fig. 5-86 shows that the entire spiral cage may detach from the concrete when the cage diameter is small (D=30 in, 762 mm). This was evident from the large tensile crack width in concrete marked by an arrow in the figure. These
high tensile damage are not observed in Fig. 5-104 in which the cage diameter is larger (D=42, 1067 mm).

5.3.5. Embedment Length of the Pipe

Very small pipe embedment length could impair the pipe-pin performance. On the other hand, excessive embedment length would be unnecessary and inefficient. The sensitivity of the load-displacement behavior of pipe-pins to this parameter is presented in Fig. 5-107. Three embedment lengths of 3D_p (42 in, 1067 mm), 4.2D_p (60 in, 1524 mm), and 5.5D_p (77 in, 1956 mm) were studied in the analyses. 4.2D_p is being used in Caltrans Design-A, two other embedment lengths were selected as extreme values to investigate the effect of this parameter. Figures 5-108 through 5-113 show the stress/strain distributions.

The pipe length did not affect the behavior before 1.25 in (31.75 mm) hinge slip in Fig. 5-107. After this point, the 3D_p embedded pipe started to rigidly tilt as opposed to the other cases in which the pipe bent. Figure 5-108 shows that the pipe did not pass the failure plane which is indicated by a dashed line and a full shear failure in column. This behavior is in contrast with that shown in Fig. 5-111 in which the pipe intersected the failure plan. As will be discussed in Chapter 6, the flexural capacity of the steel pipe contributes to the capacity (dowel effect) when it intersects the shear failure plane (as shown in Fig. 5-111). When the pipe-pin is too short, the lateral capacity does not include the participation of the pipe-pin dowel effect; thus, the pipe-pin capacity declines. The analysis showed that the ultimate capacity dropped about 205 kips (925 kN)
(approximately 13%) when the embedment length was 3Dₚ. On the other hand, if the pipe-pin is too long, no significant enhancement is expected. Figure 5-107 illustrates that the capacity improved approximately 4% when the embedded length increased from 4.2Dₚ to 5.5Dₚ. In this research, embedded length of 4.5 times the diameter of pipe was recommended for design purpose.

5.3.6. Protruded Pipe Length and the Gap between Pipe and the Can

To guarantee a moment free hinge action, the pipe has to be able to freely rotate inside the can. If the protruded pipe is too long or the side gap between the pipe and the can is too small, a moment will be developed in the pipe at the hinge throat. This moment would increase the column plastic shear and the demand on the footing. There are two sources for rotation of the pipe inside the can: flexural deformation of the pipe-pin inside the column, and rotation of top of the column under lateral loading. If the pipe-pin remains elastic, flexure of the column is the main source of rotation of the pipe inside the can. A sufficient side gap has to be provided to accommodate the anticipated rotation of the pipe. However, an unnecessarily large gap might increase the impact force once the friction at the connection is overcome. It is realized that the actual gap on the sides of the pipe may be highly variable because of inevitable lack of precision during construction and temperature movement of the superstructure.

To quantify the effect of the protruded length and the side gap on the hinge behavior four different combinations were analyzed: two side gap sizes of 0.5 in (12.7 mm) and 0.75 in (19 mm) for protruded length of 24 in (610 mm), and two side gap sizes of 0.25 in (6.4
mm) and 0.5 in (12.7 mm) for a 16-in (406 mm) protruded length. A protruded length of 24 in (610 mm) protrusion and 0.5 in (12.7 mm) gap are currently being used in practice. The results of this series of analysis are presented in Figs. 5-114 through 5-123.

Figures 5-115 to 5-123 show the stress/strain results associated with this parameter. The required length to transfer the lateral force to the pipe-pin is indicated by a double arrow in Fig. 5-115. As can be approximately measured in this figure, this length is roughly two-thirds of the pipe diameter. This implies that protruded length that was required to transfer the lateral force between the can and the pipe was less than the pipe diameter. It should be noted that a very small protrusion endangers the integrity of the structure in a case of very large vertical movement.

Figure 5-114 shows that the gap size and the protruded length did not change lateral capacities noticeably and the achieved capacities are quite similar. When the gap was 0.25 in (6.4 mm), the capacity increased less than 2% compared to the prototype model with 0.5 in (17.2 mm) gap thickness. This is because in the smaller can a larger portion of the circumference of the can was in touch with the pipe; on the other hand, in the larger can a smaller arc of the circumference was in touch with the pipe. To transfer the same lateral force, assuming with the same contact area, the contact region of the larger can and the pipe had to be taller. In this case the resultant bearing force between the can and the pipe shifted higher and increased the eccentricity of the lateral load from the surface. This increased the bending arm and reduced the capacity by very small amount.
A more important aspect of the behavior is gap closure at the top of the pipe, which occurs at approximately 2.5 in (63.5 mm) (indicated by an arrow in Fig. 5-114). It can be perceived from the slight increase in the load. This extra capacity is a result of the opposing moment that was generated at the pin. Upon gap closure at the top of the pipe, a couple develops at points of contact between the pipe and the can, thus leading to a moment in the pipe at the hinge throat. As Figs. 5-118 and 5-123 show, gap did not close when the gap size was 0.75 in (19 mm), or the pipe protrusion was 16 in (406 mm). Protruded length of 1.2D_p and side gap of 0.05D_p is suggested for new designs. This gap thickness accommodates 8.3% column drift before the top of pipe comes in contact with the can, as the calculation below shows:

\[
Drift = \frac{2 \times \text{side gap thickness}}{\text{protruded length}} = \frac{2 \times 0.05D_p}{1.2D_p} = 0.083
\]

5.3.7. Hinge Throat Thickness and Column Flexibility

The hinge throat is an unreinforced concrete with a diameter that is less than the column diameter (Fig. 5.1). The concrete capacity at the hinge throat is enhanced by the confinement from the surrounding mass of concrete in the column if it is located on the column or if the hinge throat is a part of the cap beam. The confining effect is less for thicker hinge throats and makes their edge vulnerable to crushing. Therefore, a very thin hinge throat (and the associated thin horizontal gap around it) is chosen in the original details to reduce the risk of damage on the throat edge. However, if the horizontal gap is too small, the gap will close and edge of the column touch the soffit and could lead to the spalling of the column edge at large rotations. This risk is even more if the column is
flexible (i.e. tall columns). It is, therefore, evident that the flexibility of column and the horizontal gap thickness (or hinge throat thickness) have inter-related effects and have to be studied together. Two column lengths of 8Dc (48 ft, 14.63 m) and 4Dc (24 ft, 7.31 m) were selected in combination with throat thickness of 0.5 in (12.7 mm), 1.0 (25.4 mm), and 2.0 (50.8 mm). The results associated to the study of this parameter are presented in Figs. 5-124 through 5-156.

Figures 5-133, 5-136, 5-145, and 5-147 capture local hinge deformations of the taller column with thin and thick hinge throats. The taller column was selected for this discussion because it imposed larger rotation at the top. Figures 5-133 and 5-145 are associated to the smaller diameter bearing area with 0.5 in (12.7 mm) and 2.0 in (50.8 mm) throat height, respectively. The same is true for Figs. 5-136 and 5-147, but for the larger diameter bearing area. Damage in the column edge is apparent in Figs. 5-133, and 5-136 as specified by ovals due to contact of the column edge with the soffit. Figures 5-145 and 5-147 show the intensive concrete cracking when the throat is thick because of the lack of confinement. It can be concluded that by reducing the hinge throat thickness, the risk of damage to the edge of column increases. Conversely, when the hinge throat is thick, the extent of damage to the hinge throat itself increases.

Figures 5-124 to 5-126 show the load displacement curves. It is clear that the flexibility of the column had minor impact on the ultimate capacity of pipe-pin. The maximum capacity reduction caused by column flexibility was for the 2-in (50.8 mm) hinge throat and was approximately 9%. The capacity of the pipe-pin associated with 0.5-in (12.7-mm), 1.0-in (25.4-mm), and 2.0-in (50.8-mm) throats was 1500 kip (6672 kN), 1525 kip
(6783 kN), and 1460 kip (6494 kN), respectively. These figures demonstrate that hinge throat thickness did not affect the capacity significantly. However, thicker throat increased the extent of damage to the edge of throat.

The analysis confirmed that the hinge gap closes and the edge of the column might be damaged for very small throat thickness. The damage at the edge of column is preferable for the designers because it is visible and repairable, but there is no access to the hinge throat because of the thin gap.

5.3.8. **Exterior Can Thickness**

Three can thicknesses of 0.5 in (12.7 mm), 0.75 in (19 mm), and 1.0 in (25.4 mm) were studied in the models. The first thickness corresponds to the value of the prototype detail. Figure 5-157 shows the load displacement curves and 5-158 to 5-163 present the stress/strain distributions. The load-deformation curves were identical for all the three can thicknesses and no improvement was observed by increasing the thickness of the exterior can. The results indicate that the failure of pipe-pins is independent of the can thickness. A practical thickness of 0.5 in (12.7 mm) is proposed for design of the can.

5.3.9. **Pipe Thickness and Diameter**

The flexural capacity of the pipe contributes to the lateral load capacity of the pipe-pin. Pipe thickness and diameter are two of the parameters that can be changed in the design process.
Three pipe thicknesses of 1.0 in (25.4 mm), 1.25 in (31.75 mm), and 1.5 (38.1 mm) were studied for an outer pipe diameter of 14 in (356 mm). Then the pipe diameter was changed to 12 in (305 mm) and 16 in (406 mm) while the pipe thickness remained 1.25 in (31.75 mm). The length of the pipe was changed to keep the ratio of the embedment to diameter constant. Figure 5-164 and 5-165 present the load-deformation results. The capacity increased by 5% when thickness increased from 1.0 in (25.4 mm) to 1.5 in (38.1 mm). By changing the diameter from 12 in (305 mm) to 16 in (406 mm), the capacity increased by 11%. This observation can be explained considering the fact that increasing the pipe diameter increases the flexural capacity more than increasing the pipe thickness does (the flexural capacity is correlated with the thickness and cube of the diameter). As will be explained later, it was recognized that failure of the pipe-pin hinge is associated to the flexural yielding of the pipe. Formulation presented for the pipe-pin in the next chapter explains how the flexural capacity of the steel pipe affects the pipe-pin capacity.

The results of these analyses also demonstrated that the current design method which is only based on the shear capacity of the pipe section is not representative of the pipe-pin behavior. For example, a 38% increase in the pipe gross cross sectional area only increased the capacity by 11%. This is because the pipe-pin capacity depends on several other parameters than the pipe gross section area.

5.3.10. Spiral Around the Exterior Can

The original pipe-pin connection detail developed by Caltrans incorporates a spiral around the can in the superstructure. The spiral around the can was removed in one of
the models to study its effect. Results of this analysis are presented in Figs. 5-178 to 5-184.

Figure 5-178 demonstrates that removing the extra reinforcement around the exterior can did not affect the pipe-pin capacity at all. This is because the large body of concrete surrounding the can provides large confinement for the compressed concrete against the can and the spiral has no additional effect. The capacity drop at very large displacements marked by an arrow is because the can slipped inside the concrete in the absence of the spiral. The tensile damage that is circled in Fig. 5-179 demonstrates this effect. To secure the can in place when it is subjected to several cycles of loading, it is suggested to weld six studs around the top of the can to guarantee the full anchorage inside the concrete.

5.3.11. Studs on the Pipe

Welding the studs on the pipe is costly and time consuming; therefore, removing these studs can facilitate the construction. The base model used in the parametric study did not include the studs. To study the importance of the pipe studs, they were added to one of the models using nonlinear embedded beam elements. The studs in the original design were Φ0.75@5 in (Φ19@127 mm) welded to four sides of the pipe. The results are presented in Figs. 5-185 to 5-191.

Figure 5-185 compares the load-displacement curves with and without studs. There was approximately 6% capacity enhancement when the studs existed. Considering the
insignificant improvement in the capacity and the construction cost of the studs, it is recommended that no studs be used on the pipes.

5.3.12. Cyclic Loading

All of the parametric analysis was done under a unidirectional monolithic loading. The damage imposed to the pipe-pin hinge might reduce the capacity under cyclic loading. To determine if the capacity of the pipe-pin is affected by cyclic loading, two different cases were analyzed. In the first case the load was reversed soon after yielding of the pipe-pin, and in the second case the load was reversed when extensive plastic deformation occurred.

Figures 5-192, and 5-193 show the load-deformation curves. They demonstrate that the pipe-pin recovered its strength in the reverse load in both cases. For the elastic cycle, the load was 1214 kip (5400 kN) and 1304 kip (5800 kN) at approximately 0.9 in (22.86 mm) displacement for the smaller and larger bearing areas, respectively. During the return cycle the load reached 1288 kip (5730 kN) and 1488 kip (6618 kN) at the same opposite displacement for the smaller and larger bearing areas, respectively. Therefore, no strength reduction was noted when the near elastic cycle was applied. The trend was slightly different for the plastic cycle. The maximum load reached 1525 kip (6783 kN) for both bearing area diameters, but in the reverse cycle it reached 1433 kip (6374 kN), which is 6% lower. The shape of the cyclic load-displacement curve depends on the level of the yielding the pipe-pin has experienced. If extensive flexural yielding has occurred in the pipe, the pipe needs to bend back under the reversed load. This is the reason for
the smooth slope of the return curve in Fig. 5-193 and is called the Baushinger effect and [Chen, 1988].

5.3.13. Column Nonlinearity

Pipe-pins are designed as capacity-protected members and are supposed to remain elastic and let the column yield and dissipate the earthquake energy. However, the large rotation at top of the column might affect the pipe-pin performance. To determine if the plastic rotation of the column affects the pipe-pin behavior, the column was modeled to have yield and ultimate lateral load capacity of 800 kip (3558 kN) and 960 kip (4270 kN), respectively. These values were obtained from analysis of the prototype column section containing 2% longitudinal reinforcement.

Figure 5-194 shows the force-slip and force-rotation curves. It is clear that when the column yielded, the pipe-pin hinge slip remained approximately the same as the pre-cracking slip. However, the force-rotation curve shows that the rotation kept increasing while the force remained constant. This proves that rotation at the column top did not impair the capacity of the pipe-pin.

These figures reveal another important fact. The maximum lateral load reached 1130 kip (5026 kN) despite the fact that ultimate plastic shear was 960 kip (4270 kN). This extra capacity comes from the pipe-pin flexural engagement inside the can at large column rotations. This was discussed in Sec. 5.3.6 and is due long pipe protruded length leading to double contact between the pipe and the can and the development of a moment in the pipe at the hinge throat. The rotations at which the pipe was locked inside the can are
marked by two arrows for the larger and smaller bearing diameter. The plastic shear from the pipe plastic moment can be calculated to be 98 kip (436 kN) by dividing the plastic moment of the pipe section by the length of column. The remainder of the difference between the lateral load and lateral capacity is due to the P-delta effect.

5.3.14. **Shape of the Column**

Some bridge column sections are square, but the longitudinal reinforcement is placed in a circular shape. The same pipe-pin detail was studied in a square column with the side dimensions equal to diameter of the circular column in the base model. The pipe-pin response in a square column was comparable with that of the circular one as shown in Fig. 5-195, but because of the larger area of the surrounding concrete in square column, the shear capacity of the section was larger and correspondingly, the capacity of the pipe-pin was larger. Based on the analysis, the capacity of the pipe-pin in the square column was 5.2% larger than the capacity of a similar size circular column.

5.4. **Concrete Strength**

To investigate the effect of concrete compressive strength, it was changed from 4.5 ksi (31 MPa) to 7 ksi (48.3 MPa) in the model. The compressive and tensile material models changed as explained in detail in Chapter 4. Figures 5-196 to 5-202 show the effect of change in concrete strength. As Fig. 5-196 shows, concrete strength is mostly effective in increasing the cracking force as marked by the arrow. Cracking force increases from 1240 kip (8550 MPa) to 1420 kip (9791 MPa) for smaller bearing area, and from 1320 kip (9079 MPa) to 1570 kip (10879 MPa) for larger bearing area.
(9101 MPa) to 1380 kip (9515 MPa) for larger bearing area. The main reason for improvement in cracking load is larger tensile capacity of concrete.

5.5. Summary of the Observations

In order to determine the general mechanism or mechanisms that control the behavior of pipe-pins under lateral loading, pattern and location of the cracks and plastic deformations were carefully studied in FE models. A summary of the findings is listed below:

- The lateral load-slip response is approximately linear elastic up to the threshold that lateral load overcomes the friction resistance.

- After the friction is overcome, the column snaps back and the steel pipe hits the steel can. After this point, the pipe comes in contact with the steel can; thus, lateral strength is the contribution of the friction force and mechanical engagement of the pipe-pin.

- Partial bearing failure occurs in concrete on the edge of the hinge throat when the column starts tilting and the axial bearing pressure shifts to the opposite side of the hinge throat.

- Bearing stresses between the exterior can and the pipe locally crushes the concrete against the can in bearing. The exterior can also deforms into an oval shape.
Shear and flexural stresses keep rising on the pipe as the lateral force increases. Approximately at the same time with beginning of the flexural yielding of the embedded pipe, a vertical crack forms on the sides of the pipe normal to the direction of loading. The bottom of this crack propagates towards the column surface in an angle of approximately 45 degrees similar due to diagonal tension. This failure plane is numbered as (1) in Fig. 5-203a and also can be recognized in the FE tensile cracking plot of Fig. 5-203b.

Under large axial loads, a major portion of the lateral load is carried through friction. Horizontal friction force on the top surface of the column opens leads to a diagonal tension crack that spreads through the width of the column. This crack is marked by (2) in Fig. 5-203a. Figure 5-203b illustrates that both the aforementioned cracks could potentially form in a pipe-pin connection.

The diagonal cracking of the column leads to bending of the pipe and potential flexural yielding.

5.6. Failure Mechanism of the Column Pipe-Pin Hinges

To find a better insight about the structural behavior of the pipe-pins the total resistance should be divided into its components. The lateral strength of the pipe-pin has two sources, 1) the force carried by mechanical engagement of the pipe inside the can, and 2) the friction between horizontal concrete surfaces around the pipe. The failure associated with the pipe resistance is expected to be in the form of crack mechanism (1) (Fig. 5-203). The friction component tends to lead to the crack mechanism (2). In other words,
what limits the friction force is the crack mechanism (2) not only the friction coefficient between concrete surfaces.

Mechanism (2) always results in a larger capacity because it involves a larger concrete area and passes trough more lateral reinforcing steel. The lateral capacity associated to the crack mechanism (2) is the upper limit for the lateral capacity of the pipe-pin. When the axial load is very small, or the surfaces are very smooth, the friction component is too small to cause such crack mechanism (2).

The pipe-pin failure in either mechanism can be imagined as partial shear failure of the column in which the shear crack does not cover the entire width of the column. This conclusion makes the importance of the shear reinforcement obvious. It has to be noted again that this shear cracking mechanism coincides with the flexural yielding and bending of the pipe.

Making use of the data generated by a large number of FE analyses, the effect of different parameters on the pipe-pin lateral capacity was numerically studied. As a part of this research project, a practical design method was developed for the pipe-pin two-way flexural hinges based on the observed mechanisms of failure. This method is presented in the next chapter.
6. DESIGN PROCEDURE

6.1. Introduction

In this chapter the details of the proposed design method are presented. The method that is currently being used to design pipe-pins hinges was modified based on the experimental and analytical results. In this chapter, observed physical behavior is translated to mathematical formulation in order to develop a practical design procedure.

The existing pipe-pin column hinges have been designed only based on the pure shear failure mode in which gross section area of the steel pipe and steel strength are the only determining parameters. The capacity based on the current design method is

\[ H_u = \varphi(0.6 F_u A_{pipe}) \]

with a strength reduction factor of 0.75.

In Chapter 5 the effect of different variables on behavior and capacity of pipe-pin column hinges was discussed. It was demonstrated that the capacity of pipe-pins hinges depends on many other variables besides the cross section area of the pipe. Parameters such as the transverse reinforcement of the column (column spiral) were found to be very important.

It was explained in Chapter 1 that pipe-pins were originally designed as two-way column hinges, but they also have been employed as shear keys [Caltrans, 2008]. The only physical difference between pipe-pin column hinges and pipe shear keys is the expected mode of failure in the surrounding concrete. Chapter 5 demonstrated that shear cracking of concrete in the column is the dominant mode of failure in column hinges, while the
push-off experiments proved that if the steel pipe is surrounded by a large body of
concrete (i.e. in pipe shear keys), the bearing failure of the concrete against the pipe or
pure shear failure of the pipe determines the limit state.

A general design procedure for the pipe-pin column hinges is first discussed in this
chapter followed by introducing a simple design procedure for pipe shear keys. The
chapter continues with presenting a simplified version of design method and ends with a
comparison of the calculated capacities using the current and the proposed design
methods. Numerical example of the design method is presented in Appendix E.

**6.2. Formulation of Pipe-Pin Connection Behavior**

Plastic hinging of the embedded steel pipe was observed in all of the parametric study
cases. The same mode was also noticed in PF-1 and push-off specimen tests.

To formulate the behavior when a flexural plastic hinge forms at a certain depth, free
body diagram of Fig. 6-1 can be considered in which $L_1$ is the nominal depth of the
flexural plastic hinge measured from concrete surface. Shear force is zero at the location
of the maximum moment according to the theory of “beam on elastic foundation”.
Equilibrium of the horizontal forces in Fig. 6-1 results in Eq. 6-1. This equation
expresses that integral of the distributed resisting force, from the concrete surface to the
plastic hinge depth, is equals to total lateral load.

$$H_u = D_e \int_0^{L_1} f_c^*(y)dy$$

Eq. 6-2 gives the desistance between plastic hinge and resultant resisting force.
\[ \bar{y} = \frac{D_p}{H_u} \int_0^{L_1} f_c^*(y) y \, dy \]  

(6-2)

\( f_c^*(y) \): Distribution of the resisting bearing force against the pipe

\( D_p \): Pipe diameter

Moment equilibrium about the center of the plastic hinge leads to:

\[ M_p = H_u (L_1 + e - \bar{y}) \]  

(6-3)

Assuming uniform bearing stress distribution, \( H_u = D_p f_c^* L_1 \) and \( \bar{y} = 0.5L_1 \). Simplifying Eq. 6-1 for uniform distribution and substituting in Eq. 6-3 gives the depth of the plastic hinge:

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

(6-4)

Using \( L_1 \) from Eq. 6-4 the lateral load capacity of the pipe-pin is:

\[ H_u = L_1 D_p f_c^* \]  

(6-5)

### 6.3. Design Procedure

#### 6.3.1. Pipe-Pin Column Hinges

The procedure presented here is to be used for iterative design of the pipe-pin two-way column hinges to satisfy the lateral load capacity requirement when subjected to extreme lateral loads. In the proposed method, first the “reference lateral load capacity”, \( H_o \),
associated to the cracking mechanism (1) (as explained in Chapter 5, Fig. 5-203) is estimated. Then the “upper bound shear capacity”, \( H_{cr} \), associated to the cracking mechanism (2) is obtained under the maximum effective axial load. Finally, the nominal capacity of the pipe-pin is obtained by interpolating between \( H_{o} \), and \( H_{cr} \) using the existing level of axial load (Fig. 6-2). The ultimate design capacity accounts for the reduction of the nominal capacity due to the impact that is resulted from the sudden slippage after the friction force at the connection is exceeded.

The design method is explained in two general steps. The first step explains how to calculate the reference lateral load capacity, and the second one concerns about upper bound shear capacity and interpolation process.

**Step 1)** Determining the reference lateral load capacity, \( H_{o} \), assuming no axial load (lateral resistance of the pipe-pin when no friction and axial load effect is involved). Figure 6-3a shows the distribution of the applied and resisting forces on the pipe-pin when cracking mechanism (1) occurs. After finding the resisting forces against the pipe, the concept introduced in Section 6.2 is used to find the reference lateral capacity.

**1a)** Estimate the flexural capacity of the in-filled pipe using moment-curvature analysis. It is important to use a suitable confinement model for the concrete inside the steel pipe. The model proposed by Sakino and Sun [2004] was used in this study. This model was developed for highly confined concrete inside the steel tube columns known as Concrete Filled Tube (CFT) columns. In the absence of a detailed analysis, the ultimate moment
could be found from Eq. 6-7. This equation was developed as part of the current project based on moment-curvature analyses of several cases:

\[ Z_{pipe} = \frac{4}{3} \left( r_1^3 - r_2^3 \right) \]  
\( (6-6) \)

\[ M_u = 1.1 f_y Z_{pipe} \]  
\( (6-7) \)

**1b) Determine depth of flexural plastic hinge, \( L_1 \) (Fig. 6-3a) using equilibrium.** To do so, values of the bearing stresses against the pipe and the can \( f_1 \) and \( f_2 \), respectively should be known. \( f_1 \) is taken to be the same as \( f_c' \) and \( f_2 \) is taken as \( 2.25 f_c' \) based on the experiments done in the first phase of this study.

\[ f_1 = f_c' \]  
\( (6-8) \)

\[ f_2 = 2.25 f_c' \]  
\( (6-9) \)

Assuming the a trapezoidal distribution for \( f_2 \) and a uniform distribution for \( f_1 \), the load eccentricity, \( e \), in Fig. 6-4 is calculated as:

\[ 0.75 L_2 f_2 = L_1 f_1 \Rightarrow L_2 = \frac{4L_1}{3 \times 2.25} = \frac{16}{27} L_1 \]  
\( (6-10) \)

\[ e = \frac{7}{18} L_2 = 0.23 L_1 \]  
\( (6-11) \)

Inserting the value of \( e \) in Eq. 6-4 gives the depth of plastic hinge.

\[ L_1 = \sqrt{(0.23 L_1)^2 + \frac{2M_p}{D_p f_c'} - 0.23 L_1} = \frac{1.37 M_u}{D_p f_c'} \]  
\( (6-12) \)
If the precise values of \( n \) and \( L \) are desired, the following detailed method may alternatively be used. First, four components of resisting force against the pipe need to be calculated based on an assumed value for \( L \). The four components of resisting force are associated to: shear and tensile resistance of concrete [Caltrans, 2006], column spiral, inner spiral, and FRP jacket (if any) when cracking mechanism (1) occurs (Fig. 6-3b, c).

The concrete splitting force is the summation of the tensile capacity to depth \( L \) with shear capacity of half of the cross section of column.

\[
V_c = 0.8A_c f_v + L_n f_i (B - D_p) \quad \text{ (kip, kN)} \tag{6-13}
\]

Where:

\[
\begin{align*}
  f_v &= \begin{cases} 
  0.095\sqrt{f'_{c'}} \, \text{(ksi)} & 
  f_i \begin{cases} 
  0.24\sqrt{f'_{c'}} \, \text{(ksi)} \\
  0.62\sqrt{f'_{c'}} \, \text{(MPa)}
  \end{cases} 
\end{cases} \\
  A_c &= \pi \left( B^2 - D_p^2 \right)/8 \\
  B & : \text{Diameter of the column} \quad \text{(in, mm)} \\
  D_p & : \text{Pipe diameter} \quad \text{(in, mm)}
\end{align*}
\]

To find the contribution of the column spiral and inner spiral equation Eq. 6-14 can be used. The first term is half of the shear capacity of steel reinforcement and the second term is associated to the legs being cut by vertical split.

\[
V_s = \beta \left( A_s f_y \frac{\pi d}{4s} + A_s f_y \frac{2L_n}{s} \right) \quad \text{(6-14)}
\]

\( A_s \) : Spiral cross section area

\( s \) : Spiral pitch

\( d \) : Diameter of spiral cage

\( f_y \) : Spiral material yielding stress
And if an FRP jacket exists, the additional shear capacity is:

$$V_{FRP} = \beta \left( t_{FRP} f_{FRP} \frac{\pi B}{4} + 2t_{FRP} f_{FRP} L_1 \right) \quad (6-15)$$

$t_{FRP}$: Thickness of FRP jacket

$f_{FRP}$: Tensile strength of FRP material, $f_{FRP} = 0.5E_{FRP}E_{u,FRP}$

In Eq. 6-14 and 6-15, the $\beta$ factor was added to take account the fact that not all the spirals reach the yielding force when cracking mechanism (1) occurs. This factor was estimated to be 0.25 based on the results of FE analyses. Therefore, the resisting force components associated to the column spirals, inner spiral, and FRP jacket can be written as Eq. 6-16, 6-17, and 6-18, respectively.

$$V_{s1} = \frac{1}{16} A_{sp1} f_{ys} \frac{\pi d_1 + 8L_1}{s_1} \quad \text{(kip, kN)} \quad (6-16)$$

$$V_{s2} = \frac{1}{16} A_{sp2} f_{ys} \frac{\pi d_2 + 8L_1}{s_2} \quad \text{(kip, kN)} \quad (6-17)$$

$$V_{FRP} = \frac{1}{16} t_{FRP} f_{FRP} (\pi B + 8L_1) \quad \text{(kip, kN)} \quad (6-18)$$

$A_{sp1}$: Column spiral cross section area \hspace{1cm} (in$^2$, mm$^2$)

$A_{sp2}$: Inner spiral cross section area \hspace{1cm} (in$^2$, mm$^2$)

$s_1$: Column spiral pitch  \hspace{1cm} (in, mm)

$s_2$: Inner spiral pitch  \hspace{1cm} (in, mm)

$d_1$: Column spiral cage diameter \hspace{1cm} (in, mm)

$d_2$: Inner spiral cage diameter \hspace{1cm} (in, mm)
\( f_{ys} \): Spiral material yielding stress (ksi, MPa)

Equation 6-19 converts the total resisting force into the equivalent uniformly distributed resisting stress. The upper limit in Eq. 6-19 controls if \( f_i \) is less than the bearing strength of concrete against the pipe. This limiting value was found in this study based on tests on six push-off specimens.

\[
f_i = \frac{V_c + V_{s1} + V_{s2} + V_{FRP}}{D_p L_1} < \begin{cases} \sqrt{f'_c} \left( \frac{2.95 - \frac{\sqrt{D_p}}{9.85}}{6.38} \right) f'_c \text{ (ksi)} \\ \sqrt{f'_c} \left( \frac{2.95 - \frac{\sqrt{D_p}}{3.35}}{2.43} \right) f'_c \text{ (MPa)} \end{cases}
\]

(6-19)

\[
n = \frac{f_2}{f_i}
\]

(6-20)

Eq. 6-21 presents the expression for \( L_1 \) using a more accurate value for \( f_i \). Iterations will be needed if the calculated plastic hinge depth is not sufficiently close to the assumed value. Instead of using this equation, the graph in Fig. 6-5 may be used.

\[
L_1 = \sqrt{\frac{M_p}{M_p + \frac{14}{27n + 2} D_p f_i}} \quad \text{(in, mm)}
\]

(6-21)

1c) Determine the reference lateral load capacity. The reference lateral capacity is equal to summation of resisting forces. Plug the \( L_1 \) and \( f_i \) into Eq. 6-22 to obtain the reference lateral load capacity.

\[
H_o = f_i L_1 D_p \quad \text{(kip, kN)}
\]

(6-22)
1d) The reference lateral capacity should not be taken larger than the pure shear capacity of the in-filled pipe, $H_{o,\text{Shear}}$, in Eq. 6-23. This empirical equation was proposed based on the experiments done on the infilled steel pipes. If the calculated reference capacity is larger than shear capacity, the pipe will fail in shear before cracking of concrete and yielding of the pipe occurs. Therefore, the smaller of $H_{o,\text{Shear}}$ and $H_o$ is the reference lateral load capacity.

$$H_{o,\text{Shear}} = \frac{2A_g f_{u}}{\pi \sqrt{3}} + \left\{ \begin{array}{l} 0.93A_{cp} \sqrt{f'_c} \quad (ksi) \\ 2.47A_{cp} \sqrt{f'_c} \quad (MPa) \end{array} \right.$$  \hspace{1cm} (6-23)

$A_g$: Cross section area of the steel pipe  \hspace{1cm} (in$^2$, mm$^2$)

$A_{cp}$: Cross section area of the concrete inside the steel pipe  \hspace{1cm} (in$^2$, mm$^2$)

**Step 2)** This step focuses on determining the upper bound shear capacity of pipe-pin column hinges, $H_{cr}$, associated with the maximum effective axial load. This failure capacity is tied to the cracking mechanism (2) as explained in previous chapter. As explained in the parametric study section, the axial load increases the shear capacity of concrete. The axial load beyond which no improvement is observed is labeled as the “maximum effective axial load”.

Figure 6-6 demonstrates the distribution of the resisting forces when cracking mechanism (2) occurs. This failure is in fact a partial shear failure of the concrete column. The following steps have to be taken in order to find $H_{cr}$. 
2a) Assuming a cracking plane starts from the back edge of the bearing area (Fig. 6-6a), the angle $\alpha_i$ in Fig. 6-6d, and the horizontal projection of the cracking plane, $A'_c$, are:

$$\alpha_i = \cos^{-1}\left(\frac{D_{bearing}}{B}\right) \quad \text{(In radians)} \quad (6-24)$$

For the circular column:

$$A'_c = \frac{2\pi - 2\alpha_i + \sin(2\alpha_i)(\frac{B}{2})^2}{2} - \frac{\pi D_p^2}{4} \quad \text{(in}^2, \text{mm}^2) \quad (6-25)$$

For the square column:

$$A'_c = \left(\frac{D_{bearing} + B}{2}\right) - \frac{\pi D_p^2}{4} \quad \text{(in}^2, \text{mm}^2) \quad (6-26)$$

$B$ : Diameter of the circular or width of the square column \quad (in, mm)

$D_{bearing}$ : Diameter of bearing area (hinge throat) \quad (in, mm)

2b) The components of the resisting force in cracking mechanism (2) are shown in Fig. 6-6. The components of the resisting force are concrete shear strength, column spiral effect, inner spiral effect, FRP jacket hoop strength, and dowel action of the steel pipe. These components are defined by primed letters to be distinguished from the ones in Step 1. The angle of cracking plane was calculated to be $54^\circ$ based on the FE analyses results.

The shear stress that has been used in Eq. 6-27 is the upper limit of the concrete shear strength suggested by Caltrans [2006].

$$V'_c = 0.8 f'_y \frac{A'_c}{\cos(54^\circ)} \sin(54^\circ) = 0.8 A'_c f'_y \tan(54^\circ) \quad \text{(kip, kN)} \quad (6-27)$$
Where: \[ f'_{v} = \begin{cases} 0.142\sqrt{f'_{c}} \text{ (ksi)} \\ 0.374\sqrt{f'_{c}} \text{ (MPa)} \end{cases} \]

\( V'_{s1} \) and \( V'_{s2} \) are integrals of the outer and inner spiral forces. To find the equation for the lateral capacity of the shear reinforcement passing the creaking plane, we can write the differential of the horizontal component of the tensile force in spirals as (Fig. 6-7):

\[
dT_s = dA_s \times f_y \sin(\alpha) \quad (6-28)
\]

\[
dA_s = \frac{A_s}{s} \, dy \quad (6-29)
\]

\[
dy = \tan(\theta)\,dx, \quad dx = \frac{d}{2} \, d\alpha \sin(\alpha) \quad \text{(Fig. 6-7)} \quad (6-30)
\]

\[
\Rightarrow dT_s = \frac{A_s f_y}{s} \tan(\theta) \frac{d}{2} \sin^2(\alpha) \, d\alpha \quad (6-31)
\]

\[
V_s = \int_{\pi - \alpha_o}^{\pi} dT_s = \frac{A_s f_y}{2s} \int_{\pi - \alpha_o}^{\pi} \sin^2(\alpha) \, d\alpha \quad (6-32)
\]

This integral is calculated for inner spiral, column spiral, and FRP jacket, if any. For the inner spiral, the integration range is the entire trigonometric circler because the cracking plane completely cuts through the inner cage. As stated in the last step, all the spiral legs do not reach to yielding simultaneously. The reduction coefficient of 0.5 in Eqs. 6-33, 6-34, and 6-35 accounts for this fact. This coefficient was obtained from the FE analyses results. The cracking angle is assumed \( \theta = 54^\circ \) for the rest of the calculations.
\[ V_{s1}' = \frac{0.5A_{pl}f_{ys}d_{1}\tan(54^\circ)}{2s_{1}} \int_{2\pi}^{\alpha_{1}} \sin^{2} \alpha d\alpha = \frac{0.34A_{pl}f_{ys}d_{1}[\cos(\alpha_{1})\sin(\alpha_{1}) + \pi - \alpha_{1}]}{s_{1}} \] (kip, kN) \hspace{1cm} (6-33)

\[ V_{s2}' = \frac{0.5A_{pl}f_{ys}d_{2}\tan(54^\circ)}{2s_{2}} \int_{-\pi/2}^{\pi/2} \sin^{2} \alpha d\alpha = \frac{0.34A_{pl}f_{ys}d_{2}\pi}{s_{2}} \] (kip, kN) \hspace{1cm} (6-34)

\[ V_{FRP}' = \frac{0.5f_{FRP}f_{FRP}B\tan(54^\circ)}{2} \int_{-\pi/2}^{\pi/2} \sin^{2} \alpha d\alpha = \frac{0.34f_{FRP}f_{FRP}B[\cos(\alpha_{1})\sin(\alpha_{1}) + \pi - \alpha_{1}]}{} \] (kip, kN) \hspace{1cm} (6-35)

The fifth component of the resisting force is the dowel action of the pipe which passes the cracking plane. \( V_{pipe} \) is the lateral load resisted by the bending of the cantilever pipe and can be found as Eq. 6-36:

\[ V_{pipe} = \frac{M_{y}}{(D_{bearing} + D_{p})\tan(54^\circ)} = \frac{1.45M_{y}}{D_{bearing} + D_{p}} \] (kip, kN) \hspace{1cm} (6-36)

The upper bond shear capacity is the summation of these five components of resisting forces as specified in Eq. 6-37.

\[ H_{cr} = V_{c}' + V_{s1}' + V_{s2}' + V_{FRP}' + V_{pipe} \] \hspace{1cm} (6-37)

2c) Determine the maximum effective axial load based on Eq. 3.21 of the Seismic Design Criteria [Caltrans, 2006]:

\[ N_{u} = \begin{cases} 1A'_{c} \text{ (ksi)} \\ 0.007A'_{c} \text{ (MPa)} \end{cases} \] \hspace{1cm} (6-38)
2d) Calculate the nominal lateral load capacity of the pipe-pin hinge by interpolating between Reference \((H_o)\) and Upper Bound \((H_{cr})\) capacities, according to the expected axial load on the column during the earthquake using Eq. 6-39:

\[
H'_n = H_o + \left( H_{cr} - H_o \right) \left( \frac{N}{N_u} \right)^{0.7}
\] (6-39)

Table 6-1 compares the results from the FEM, detailed design method, and simple design methods as explained earlier. Compared to the FEM results, the average error when the detailed and simple methods were used to determine \(H_o\) was 2.4% and 3%, respectively.

The close correlation between the results of the proposed simple method and that of FEM demonstrates that the proposed method is appropriate for design application.

2e) The lateral load capacity needs to be modified to take into account the impact effect. When the friction between the column and the bent cap is releases, the column snaps and the pipe impacts the can. The impact force depends on the natural period of the column and the gap size. The stiffer the column or the wider the gap, the larger the impact forces are. The fundamental vibration period of a cantilever element is calculated using the mode shape of Eq. 6-40 along with Eqs. 6-41, and 6-42 [Chopra, 2006] for a distributed mass cantilever system.

\[
\varphi_n(x) = \cosh(\beta_n x) - \cos(\beta_n x) - \frac{\cosh(\beta_n L_c) + \cos(\beta_n L_c)}{\sinh(\beta_n L_c) + \sin(\beta_n L_c)}(\sinh(\beta_n x) + \sin(\beta_n x))
\] (6-40)

\[
m_1 = \int_0^{L_c} \rho \left[ \varphi_n(x) \right]^2 dx
\] (6-41)
\[ K_1 = \int_0^{L_c} EI[\varphi(x)]^2 \, dx \]  

(6-42)

For the first mode of vibration, \( \beta_1 L_c = 1.8751 \). And natural period is:

\[ T = \frac{2\pi L_c^2}{3.516 \sqrt{\frac{\rho}{EI}}} \quad \left( T = \frac{2\pi}{\sqrt{K_1}} \right) \]  

(6-43)

\( \rho \) : Mass per unit length of the cantilever column (kip/in, kN/mm)

\( L_c \) : Length of the column (in, mm)

\( EI \) : Elastic flexural rigidity of the column (kip-in², kN-mm²)

The element with distributed mass may be replaced by a cantilever column with an equivalent lumped mass at the top by matching the periods of the two systems.

\[ K = \frac{3EI}{L_c^3} \quad \Rightarrow \quad m = 0.243 \rho L_c \]  

(6-44)

The energy conservation law leads to the velocity of the impact as:

\[ v = \sqrt{\frac{K \times G^2}{m}} \]  

(6-45)

\( G \) : Size of the side gap between pipe and can (in, mm)

Assuming that the impact duration is one quarter of a full cycle (the oscillator goes from zero force to maximum in quarter of a full cycle), the law of conservation of momentum gives the impact force by Eq. 6-46. Substitutions and simplification leads to.

\[ F_{\text{impact}} = \frac{mv}{0.25T} \]  

(6-46)
As a capacity protected member, the pipe-pin is intended to remain elastic while the column undergoes large plastic deformations. To accomplish this goal, the lateral capacity of the pipe-pin should be larger than the plastic shear of the column.

For design purpose, the nominal capacity needs to be reduced by a reduction factor to guarantee that the pipe-pin remains elastic. The nonlinear behavior of the pipe-pin coincides with cracking of the concrete that surrounds the pipe. Table 6-1 shows the ratio of the cracking loads to the ultimate capacities obtained from different parametric analyses. This ratio ranges from 0.75 to 1.00, depending on the reinforcement and detailing of the reinforcement around the pipe. Therefore, a capacity reduction factor of 0.75 is recommended to ensure that the pipe-pin connection remains elastic (i.e. uncracked). To account for the impact effect, the impact force should be subtracted from the factored capacity. Therefore, the ultimate design capacity can be determined from Eq. 6-48.

\[ H_u = \phi(H_u) - F_{\text{impact}} \]  
\[ \phi : \text{Capacity reduction factor} \]

6.3.2. Pipe Shear Keys

The procedure that is presented here is to be used for design of the pipe shear keys to satisfy the lateral load capacity requirement when subjected to the extreme lateral loads. The basic difference of pipe shear keys with the pipe-pin column hinges that was
formulated in Section 6.3.1 is the governing modes of failure. The shear keys are typically located in abutments or bent caps; thus, cracking of the concrete is not the expectable mode of failure. In this case, capacity is mostly limited by either the bearing failure of the concrete or pure shear failure of the infilled pipe. The flexural capacity of the pipe is found from Eqs. 6-6 and 6-7 similar to the pipe-pin hinges.

The equivalent uniform bearing strength of concrete against the pipe is calculated based on empirical Eq. 6-49. If the inner spiral is absent, use 90% of the values from this equation.

\[

c_c^* = \left( \frac{\sqrt{f_c'}}{2.43} \right) \left( 2.95 - \frac{3D_p}{3.35} \right) f'_c \text{ (ksi)}
\]

\[

c_c^* = \left( \frac{\sqrt{f_c'}}{6.38} \right) \left( 2.95 - \frac{3D_p}{9.85} \right) f'_c \text{ (MPa)}
\]

Using Eqs. 6-4 and 6-5, the capacity of the pipe shear keys excluding the effect of the friction is:

\[
H_o = D_p c_c^* \left( e^2 + \frac{2.2f_c'Z_{pipe}}{D_p f_c^*} - e \right) < \frac{2A_p f_u}{\pi \sqrt{3}} + \left( 0.93A_p \sqrt{f_c'} \right) \text{ (ksi)} + 2.47A_p \sqrt{f_c'} \text{ (MPa)}
\]

\(e\): Eccentricity of the resultant load from surface, conservatively \(e = 0.3D_p\)

The upper limits are the pure shear capacity of the infilled pipe that was discussed in Section 6.3.1. The effect of friction should be added to this force to arrive at the nominal capacity. After applying the strength reduction factor, the final form of the ultimate capacity associated with bearing or pure shear failure becomes:
\[ H_u = \varphi (H_o + \mu N) \quad \text{(kip, kN)} \quad (6-51) \]

\( \varphi \): Capacity reduction factor

\( \mu \): Friction coefficient of concrete surfaces based on ACI-318 design code

\( N \): Axial load on the pin

### 6.4. SDC Format Simplified Design Procedure

The method that was introduced in Section 6.3.1 was simplified for the pipe-pin hinges that follow guidelines that are presented in the next section following the seismic design criteria (SDC) [Caltrans, 2006] format.

- **Nominal Lateral Load Capacity of Pipe-Pin Hinges**

The seismic lateral load demand shall be based on the overstrength shear associated with the overstrength moment [SDC, Section 4.3]. The lateral capacity shall be conservatively based on the nominal material strengths.

\[
\varphi H_n > V_o + F_{impact} \quad \varphi = 0.75 \quad (6-52)
\]

\[
H_n = H_o + (H_{cr} - H_o) \left( \frac{N}{N_u} \right)^{0.7} \quad (6-39)
\]

\[ F_{impact} = 1.9 \frac{G \times EI}{L_c^3} \quad (6-47) \]

- **Reference Lateral Load Capacity**

\[
H_o = 1.17 \sqrt{M_u D_p f_c'} + \begin{cases} 
2A_p f_u + & \text{ksi} \\
0.93 A_{cp} \sqrt{f_c'} & \text{MPa} \\
2.47 A_{cp} \sqrt{f_c'} & \text{MPa}
\end{cases} \quad (6-53)
\]
\[ M_u = 1.45 f_y \left( r_1^3 - r_2^3 \right) \] 

(6-54)

- **Upper Limit Lateral Load Capacity**

\[
H_{cr} = \begin{cases} 
\text{Factor 1} \times \left( 0.16 A_c \sqrt{f_c'} + \frac{A_{sp1} f_{ys} d_1}{s_1} \right) + \frac{A_{sp2} f_{ys} d_2}{s_2} + \frac{1.45 M_u}{D_{bearing} + D_{p}} & (\text{ksi}) \\
\text{Factor 1} \times \left( 0.4 A_c \sqrt{f_c'} + \frac{A_{sp1} f_{ys} d_1}{s_1} \right) + \frac{A_{sp2} f_{ys} d_2}{s_2} + \frac{1.45 M_u}{D_{bearing} + D_{p}} & (\text{MPa}) 
\end{cases}
\]

(6-55)

Factor 1 = 0.45 \frac{D_{bearing}}{B} + 0.6

(6-56)

For the circular column: \[ A_c = \frac{\pi}{4} \left( B^2 - D_p^2 \right) \]

(6-57)

For the square column: \[ A_c = B^2 - \frac{\pi D_p^2}{4} \]

(6-58)

- **Maximum Effective Axial Load**

\[
N_u = \begin{cases} 
\text{Factor 1} \times A_c & (\text{ksi}) \\
0.007 \times \text{Factor 1} \times A_c & (\text{MPa}) 
\end{cases}
\]

(6-59)

6.5. **Detailing Recommendations**

The lessons learned from the numerous parametric analyses led to a series of detailing recommendations. The background information for each detailing suggestion is briefly restated to justify the recommendation.

a) Analytical parametric studies showed that short embedment length might lead to rigid body rotation of the pipe in the column. In contrast, no capacity
improvement was observed when the pipe length was increased beyond certain limits. An embedment length of $4.5 \, D_p$ is recommended for the pipe.

b) Excessive protruded length could be detrimental to the hinge action due to double curvature bending of the pipe inside the can and the resulting moment at the hinge throat, which is undesirable. A short protrusion length could compromise the bridge integrity when large uplifting forces is anticipated. A length of $1.2 \, D_p$ for the can ensures a stable and constructible detail without leading to double-curvature bending.

c) FE modeling of the detail confirmed that the capacity of the pipe-pin connection is independent of the can thickness. Using a practical minimal thickness of 0.5 in (12.7 mm) is sufficient for the normal size pipe-pins.

d) The massive concrete in the superstructure provides sufficient confinement around the exterior can, thereby eliminating the need for the supplemental confinement spiral around the can.

e) Four to six studs welded to the upper part of the can are recommended to stabilize the can inside the cap beam.

f) Analytical parametric studies showed that the studs on the pipe-pin increase the capacity only by an insignificant amount. Eliminating the studs simplifies construction and hence is recommended.
g) It is understood that the small hinge throat of 1 in (25 mm) might lead to the chipping of concrete at the edge of the column under large drifts, and that this is a minor damage that can be easily fixed after earthquake.

h) Analytical parametric studies showed that increasing the diameter of the inner spiral cage would increase the capacity and lead to a more stable result because of the higher number of legs that intersect the shear failure plane. Furthermore the anchorage of the spirals improves when the diameter is larger. An inner spiral diameter of $3D_p$ is recommended.

i) Based on the observed failure mechanism, it is found that the spiral at the top of the column contributes significantly to the capacity because the shear failure plane intersects the spiral. Therefore, the column spiral at the top of the column should be designed to achieve the target strength of the pipe-pin connection. Using the minimal confinement steel on the basis that the column top is a pin subjected to a small moment jeopardize the safety of the pipe-pin connection.

j) The recommended gap size of $D_p/20$ between the pipe and the can would accommodate the rotation demand of the pipe-pin inside the exterior can. This gap size accommodates approximately 8% drift for protruded length of $1.2D_p$ before the gap between the pipe and the can closes.

Figure 6-8 shows the detail of the typical pipe-pin when considering the above mentioned detailing recommendations. This detail can be known as a typical pipe-pin hinge design.
6.6. **Comparison with PF-1 Test Data**

The proposed procedure was utilized to calculate the lateral capacity of the pipe-pin hinge that was used in PF-1. The reinforcement and material properties of this model are introduced in Chapter 4. From the presented formulation, the nominal lateral capacity of the pipe-pin was obtained 176.5 kip (785.1 kN). This value is comparable with the plastic shear of the column that was 180.4 kip (802.4 kN). It shows that the design formulation could calculate the capacity with approximately 2% error.

6.7. **Capacity of Pipe-Pin Hinges in Bay Bridge Approach Ramps**

The nominal capacities (un-factored strength) of the pipe-pin column hinges that are used in San Francisco Bay Bridge replacement project were calculated based on the method introduced in this chapter. Results are presented in Table 6-2. The table lists the column sizes and reinforcing details. The capacities were calculated using the proposed method and compared with the results from current design method, which is based on the pure shear capacity of the steel pipe.

The theoretic results show that the lateral load capacity is very sensitive to the amount of column spiral, and that using the capacity that is obtained based pure shear failure of the pipe overestimate the strength by a significant margin in several bents. The ratio of the capacity based on the current practice and the proposed method ranged from 0.59 to 1.24.
7. IMPROVING PIPE-PIN CONNECTIONS

7.1. Introduction

Pipe-pin connections have the potential to be modified for better performance. These connections can also be combined with isolating or damping devices. Several details to enhance pipe-pin connections are presented in this chapter. Some of the options that pipe-pin connections offer for new bridge systems are also presented in this chapter.

7.2. Pipe-Pin Hinge with Tapered Hinge Throat

Crushing of concrete on the edge of the hinge throat and the column is inevitable in the current detail (Fig. 7-1). This potential damage was observed in both the experimental and analytical models. To reduce the extent of damage on the top edge of the column, the thickness of the hinge throat may be increased. However, thick gaps make the throat edge more vulnerable to damage because of the lower confinement. In the current design, the throat thickness is relatively small to minimize damage to the throat, where there is no practical access for repair.

A tapered hinge throat is suggested in this study to reduce the risk of damage to the edge of column and to increase confinement on the throat edge. Figure 7-2 shows the modified detail and the deformed shape under lateral loading. The required throat thickness would be calculated based on the design drift ratio of the column.
The other drawback of existing pipe-pin detail is the large impact forces between the pipe and the can after the superstructure slips relative to the column. An easy practical solution to this problem is to incorporate a stiff rubber washer in the lower portion of the can to absorb the impact (Fig. 7-3). Filling the entire height of the side gap may impair the hinge action of the connection.

7.3. **Pipe-Pin as Seismic Isolator**

The effectiveness of seismic isolation in reducing the force demand on structures is well understood. In an integral bent cap, elastomeric bearings or sliders are placed between the columns and the superstructure. A backup system should be provided to save the integrity of the structure in case the isolation system fails. The pipe-pin connection can perform as a stop key in isolation systems when lateral displacements exceed certain limits. Figure 7-4 shows the use of an elastomeric bearing pad in place of concrete hinge throats. This figure also shows the hinge when the pad deforms in shear under lateral loading.

To investigate the performance of this detail, the FE model of prototype detail (Ch. 5) was modified by incorporating a 1.0-in (25-mm) thick linear elastic material as shown in Fig. 7-4. Figure 7-5 shows the force-displacement results in terms of hinge slip and hinge rotation. The pad was modeled with a modules of elasticity of 0.15 and 0.3 ksi (1.03 and 2.06 MPa) and Poison ration of 0.5. The capacity dropped approximately 25% when the bearing pad was incorporated in the connection because friction between concrete surfaces was eliminated. The capacity of the isolated pipe-pin hinge can be
improved by increasing the inner and column spirals in the hinge area, or by using a stiffer elastomeric pad as illustrated in Fig. 7-5.

7.4. Pipe-Pin Connections with Damping Devices

It is possible to add energy dissipation capability to pipe-pin connections by taking advantage of the sliding that occurs at the hinge. This could limit the column base damage. One possible configuration of an energy damping device is shown in Fig. 7-6. In this configuration a series of steel full or half-ring mild steel elements are welded to the pipe and the can to perform as yielding dampers. These dampers yield in flexure, torsion, and shear when sliding occurs.

An FE model of the detail was developed to investigate its behavior. Two models with 16 and 32 half-rings with square section of 1 x 1 in (25.4 x 25.4 mm) were studied. These damper elements were added to the prototype detail that was used in parametric studies. The can diameter was increased to provide 4-in (101.6-mm) of gap around the pipe and accommodate the rings. The can thickness was also increased to 1 in (25.4 mm) to reduce deformations in the can. A 1-in (25.4-mm) elastomeric bearing was assumed at the hinge throat to allow for sliding. The models were loaded to a full cycle of ±2 in (±50.8 mm) displacement. Figure 7-7 shows the force-deformation curves for these two cases. The response was very stable and the area under the curves, which is a measure of damped energy, was large. The small rotation at top of the pipe in this figure demonstrated that column did not deform significantly and that most of lateral displacement was due to the yielding in the metallic dampers.
Figure 7-8 shows the model. The upper part of the pipe was removed in the figure to expose the rings. Figure 7-9 shows the deformed shape at the maximum displacement. The distortion and bending of the rings are evident. For this demonstration case, prismatic rings were used, but the behavior may further be enhanced if the cross section of the ring varies according to the moment in the ring.

This damper was utilized in the analytical model of the two-column pier (Ch. 4) to evaluate its effect on performance of the structure. The lateral yield strength of the damper was assumed to be 25 kip (111.2 kN) and the gap thickness was 1.5 in (38.1 mm). Figures 7-10 through 7-12 compare the measured displacement histories of the cap beam and top of the columns during the last four runs with the analytical results assuming the dampers are incorporated in the pier. Figure 7-10 illustrates that by utilizing the damping devices, the maximum displacement of the bent cap was reduced from 5.32 in (135.1 mm) to 4.34 in (110.2 mm). However, as Figs. 7-11 and 7-12 show, the maximum displacements at the top of the columns were reduced from 4.95 in (125.7 mm) and 4.92 in (124.9 mm) to 2.96 in (75.2 mm) and 2.92 in (74.2 mm) for RC column and FRP tube column, respectively. Reduction in displacement demands in columns due to the dampers was approximately 40%. It is clear that the additional damping can significantly reduce the ductility demands of the columns.

7.5. Application of Pipe Pins in Accelerated Bridge Construction

Pipe-pin connections eliminate the need for any wet joint to connect the superstructure to prefabricated columns or other elements of the superstructure. This detail allows for
placing the precast deck on the previously constructed columns. This distinct characteristics of pipe-pin connections offers many options for accelerated bridge construction (ABC).

7.6. Uplift Issue in Pipe-Pin Hinges

Because pipe-pin connections are incapable of transferring any tensile force, the application of these hinges when uplift forces exist may be of concern. Tensile forces may exist in bridge columns for a variety of reasons. In this section the issue of uplift in bridges that incorporate pipe-pin connections is discussed and a modified detail is proposed for the cases in which uplift is inevitable.

The main source of uplift in bridges is the overturning moments under lateral loads. The overturning moments in narrow and tall bridges generate large uplift forces when $\Delta N$ in Eq. 7-1 becomes larger than gravitation axial load of the column.

$$\Delta N = \frac{Ve}{s} \quad (7-1)$$

$\Delta N$: Overturning axial load on the column

$V$: Lateral load on the bent

$e$: Distance between the center of gravity of superstructure and the hinge surface

$s$: Center-to-center distance between columns

Figure 7-13 shows the elevation of the bridges for two cases: one when the hinges are located at top of the columns and other when the hinges are at column bases. When the column is connected to the superstructure by pipe-pin hinges the overturning moments
are considerably smaller than those of the columns with pinned bases (Fig. 7-13b). Therefore when the column is pinned at the top, the bridge is less vulnerable to uplift.

The longitudinal loading of the bridge may also result in uplift in columns. In a bridge with monolithic column-superstructure connections (Fig. 7-14b), the moments at the columns can alter the axial loads on some of the columns when the superstructure deflects longitudinally. In a bridge that incorporates pipe-pin hinges at the top of the columns, no moment is theoretically generated at the joints and the superstructure does not bend under longitudinal loads (Fig. 7-14a) and the existing axial load in the columns remains constant.

Another cause that may result in uplift is the vertical component of earthquake excitation. This effect needs more detailed analyses. However, it should be noted that to disconnect the superstructure from the column the entire superstructure would need to move up 12~16 in (304.8~406.4 mm) and that would require a relatively large spike in the vertical component of earthquake.

When the analyses show that the uplift and disengagement of the connection is an issue, a restrainer rod can be incorporated in the connection to prevent the vertical movement of the superstructure (Figure 7-15). A single high strength rod or a bundle of post tensioning cables can be utilized as restrainer. It is important not to fill the pipe with concrete to allow for unrestrained bending of the restrainer when the connection slides. To compensate for the effect of concrete inside the pipe, a larger pipe thickness can be used. The rod does not need to be post tensioned.
8. SUMMARY AND CONCLUSION

8.1. Summary

Two-way hinges have been used in reinforced concrete bridge column for many years. These hinges are implemented as the connecting link between column and the bridge superstructure or the footing. They allow for relative rotation between the components and are used to prevent excessive flexural stresses from passing through the connection. There are two types of modern two-way hinges in concrete bridges: one made with steel bars and the other newer detail made with steel pipes. The latter detail is known as telescopic pipe-pin hinge and was developed by Caltrans bridge designers.

The primary objective of this study was two folds: (1) to investigate the seismic performance of the current detail of pipe-pin hinges and propose necessary modifications and (2) to develop a reliable analytical method for pipe-pin hinges that reflects their actual behavior. A series of experiments were designed to study and formulate different failure modes. These experiments were followed with comprehensive finite element modeling aiming at investigating all the important design parameters of the pipe-pin hinges.

Three sets of test models were studied. The purpose of these models was to address literature gap and proof testing of a proposed design method. Six push-off specimens were constructed to study the bearing strength of concrete against the steel pipe. The
main focuses in these tests was to investigate the effect of pipe diameter and spiral around the pipe on the bearing strength of concrete compressed by laterally loaded embedded pipes. An empirical formula was proposed based on the test data and the existing relationships for reinforcing bars. In the next phase of experiments, six concrete filled steel pipes with different diameters and thicknesses were tested in pure shear. The results led to two equations, one for the yield and the other for ultimate strength of the infilled pipes under pure shear.

The experimental results from push-off specimens and a hinged single column that was tested previously were used to calibrate and verify the finite element models, which were then utilized to conduct a comprehensive parametric study of pipe-pin connections. All important parameters affecting the seismic performance of the pipe-pin hinges were investigated through the analysis of over one hundred cases using ABAQUS. The parameters included geometry, reinforcement, material properties, column prosperities, etc.

The analytical results along with experimental observations were used to identify the modes of failure under lateral loads and to incorporate them in a proposed iterative design method. The analytical results helped identify the necessary improvements in the current detail, and a typical standard detail was proposed.

A two-column 0.2-scale bridge pier was designed using the proposed design guidelines. This model was used in proof testing the modified pipe-pin detail under realistic earthquake excitation. This pier also incorporated a new column using a novel structural
system that was made up of a concrete filled fiber reinforced polymer (FRP) tube with bidirectional inclined fibers. A load-cell in the middle of cap beam allowed for measuring the base shear of individual columns. This structural model was subjected to an acceleration record from The 1994 Northridge Earthquake that was applied through seven runs and was progressively increased up to approximately two times the original ground motion. The experiment was conducted on one of the shake tables at University of Nevada, Reno Large Scale Structures Lab. The experiment was followed by analytical modeling of the pier using OpenSees. A macro model was developed for the pipe-pin hinges used in the OpenSees model.

Several modifications were proposed for variations in pipe-pin hinges to accommodate isolation and damping systems for potential future use. A brief analytical study was performed on these details as a proof of concept.

Although pipe-pin connections are designed as a column hinge, they have other applications such as shear keys and seat extenders. This study also addressed pipe shear keys based on the analytical and experimental studies reported in this document.

**Observations**

The following observations present the highlights of what was learned from the experimental and analytical results:

1. Large relative rotations can be accommodated in pipe-pin hinges without impairing their performance.
2. Two mechanisms resist the lateral load in a pipe-pin connection: friction and mechanical engagement of the steel pipe inside the can.

3. In contrast to pure shear failure mode that is assumed in the current design method, the dominant mode of failure in pipe-pin column hinges is partial shear failure of the concrete column in the hinge area. When the column lateral steel is relatively large, two other failure modes are possible: pure shear failure of the infilled pipe and bearing failure of the concrete against the pipe.

4. In pipe shear keys, the geometry of the structure does not allow for shear cracking of the concrete; therefore, the limit states under lateral loading would be the pure shear failure of the infilled pipe or bearing failure of the concrete.

5. The experimental data demonstrated that equivalent uniform bearing strength against the pipe may be taken as twice the uniaxial compressive strength of concrete. However, the local bearing strength can peak up to six times the uniaxial compressive strength of concrete.

6. Smaller pipe diameters resulted in larger bearing strength due to a better confinement of the surrounding concrete. The inner spiral around the pipe increased the bearing strength by 10% by improving confinement of concrete.

7. A very ductile response was observed when infilled pipes were tested in shear.
8. The proof test confirmed that the proposed design guideline is reliable and safe. Measured strains during the test on the steel pipe and inner and column spiral remained well below the yield strains.

9. Minor spalling was observed at the edges of the RC column in the two-column pier. This damage occurred because the hinge gap was closed and column edge came in contact with the cap beam under large rotations.

10. After the test, the cap beam was removed and status of the pipe-pin hinges was investigated. Pipes were straight, intact, and with no sign of damage. The hinge throats were ground during several cycles of loads. Small dents were detectible in the can at point of contact with the pipes.

11. The Measured bond-slip rotations were comparable for RC column and FRP tube column. The friction release in the hinge throat surfaces helped to dissipate approximately 7% of the total energy.

12. The shake table experiment demonstrated that FRP tube column system is capable of performing as a ductile, reliable structural member. No damage was detected in the plastic hinge zone of FRP tube column before tensile rupture occurred in the FRP tube.

13. Between the two columns, the RC column damped a larger portion of energy compared to the FRP tube column.
14. The rotation at the base of the FRP tube column was comparable with the bond-slip rotation in the RC column.

15. Embedment of FRP tube into the footing with a length of one and one-half times diameter of tube could provide full fixity and transfer the plastic moments.

16. The post-test analytical results matched the experimental results with good accuracy.

8.2. Conclusions

The following conclusions were made based on the experimental and analytical results presented in this document:

1. Pipe-pin connections perform as a near perfect flexural hinge while transferring shear and axial loads. This makes pipe-pin connections a more attractive detail than hinges with distributed steel bars in which some level of moment transfer is inevitable.

2. Pipe-pins can be properly designed to remain elastic, while other concrete hinges require yielding to perform as a hinge.

3. The local bearing strength of concrete may be substantially higher than the uniaxial compressive strength of concrete due to the confining effect of the surrounding concrete.

4. This study showed that the nonlinear FE analytical models can accurately approximate the modes of failure when proper element models and contact algorithms were used. Comparison of the test data and analysis results showed close correlation.
in terms of mode of failure, stiffness, and capacity for the push-off specimens and PF-1.

5. The proposed simple design method led to close estimate of the capacity of the pipe-pin column hinges that was obtained from detailed finite element analysis and experiment, with less than 5% error.

6. Using the current design method, which only accounts for pure shear failure of the steel pipe, would result in an unconservative estimate of the capacity of pipe-pin hinges.

7. Most of the detailing recommendations used for pipe-pins in the current Caltrans practice are suitable. However, results showed that no studs are necessary on the pipe, and no benefit is gained from the spirals around the can. By increasing the diameter of the inner spiral to three times the pipe diameter the pipe-pin connection capacity increases significantly. Furthermore, the protruded length of the pipe should be limited to avoid bending of the pipe in double curvature and the development of a moment at the hinge throat.

8. The column spiral contributes significantly to the capacity of pipe pin connections. The spiral should be designed specifically for the pipe pin joint, and extending the shear or confinement spiral to the hinge zone may be insufficient.

9. The maximum relative rotation that occurred between the cap beam and columns was comparable to the maximum drift ratio of the column.
10. The experimental results demonstrated that the plastic hinge length is longer in the FRP tube column compared to the RC column. This will result in larger spread of plasticity and will reduce the local strain demand on the material.

11. The pipe-pin detail provides many possibilities to be modified to isolation and damping devices.

12. When the pipe-pins are incorporated at the top of the columns, the uplift due to overturning moment is not a concern. However, when the hinge is located at the bottom of the column, overturning moment can overcome the gravity and pull the pipe off the steel can. In this case, it was proposed to use a restrainer to guarantee the integrity of the structure.
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TABLES
Table 2-1. Specification of the Push-off Specimens

<table>
<thead>
<tr>
<th>Name of the specimen pair and H-B-L in (mm)</th>
<th>Pipe size in (mm)</th>
<th>O.D. in (mm)</th>
<th>Thickness in (mm)</th>
<th>Inner spiral size and pitch in (mm)</th>
<th>Outer spiral size and pitch in (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PS1P-A, B 18-36-48 (457-914-1118)</td>
<td>3.5 xx-Stg.</td>
<td>4.0 (101.6)</td>
<td>0.636 (16.15)</td>
<td>Φ0.192@1 (Φ4.9@25)</td>
<td>Φ0.248@1.5 (Φ6.3@38)</td>
</tr>
<tr>
<td>PS2P-A, B 18-36-48 (457-914-1118)</td>
<td>3.5 xx-Stg.</td>
<td>4.0 (101.6)</td>
<td>0.636 (16.15)</td>
<td>-</td>
<td>Φ0.248@1.5 (Φ6.3@38)</td>
</tr>
<tr>
<td>PS3P-A, B 14-28-36 (356-711-914)</td>
<td>2.5 xx-Stg.</td>
<td>2.875 (73.1)</td>
<td>0.552 (14)</td>
<td>Φ0.148@0.75 (Φ3.76@19)</td>
<td>Φ0.192@1 (Φ4.9@25)</td>
</tr>
</tbody>
</table>

Table 2-2. Specification of Steel Parts in the Push-Off Specimens

<table>
<thead>
<tr>
<th>Element</th>
<th>$f_y$ ksi (MPa)</th>
<th>$f_u$ ksi (MPa)</th>
<th>Elongation %</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.5 xx-Stg. Steel pipe</td>
<td>47.8 (329.6)</td>
<td>79 (544.7)</td>
<td>33</td>
</tr>
<tr>
<td>2.5 xx-Stg. Steel pipe</td>
<td>47.6 (328.2)</td>
<td>73.4 (506.1)</td>
<td>31</td>
</tr>
<tr>
<td>W1.7 Wire Φ0.148 in</td>
<td>59 (407)</td>
<td>70.8 (488)</td>
<td>17</td>
</tr>
<tr>
<td>W2.9 Wire Φ0.192 in</td>
<td>59 (407)</td>
<td>73.2 (505)</td>
<td>10.5</td>
</tr>
<tr>
<td>W5 Wire Φ0.249 in</td>
<td>55 (379)</td>
<td>71.8 (495)</td>
<td>18</td>
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</table>
### Table 2-3. Specification of the Pure Shear Specimens

<table>
<thead>
<tr>
<th>Name of the Specimen</th>
<th>Pipe Size</th>
<th>Shear Span in (mm)</th>
<th>O.D. in (mm)</th>
<th>Thickness in (mm)</th>
<th>Pipe Steel Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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</tr>
<tr>
<td>IPS-1A</td>
<td>3.5 Std.</td>
<td>2.0 (50.8)</td>
<td>4.0 (101.6)</td>
<td>0.226 (5.72)</td>
<td>51.48 (355)</td>
</tr>
<tr>
<td>IPS-1B</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>61.8 (426)</td>
</tr>
<tr>
<td>IPS-2A</td>
<td>3.5 x-Strong</td>
<td>2.0 (50.8)</td>
<td>4.0 (101.6)</td>
<td>0.318 (8.07)</td>
<td>51.0 (351.6)</td>
</tr>
<tr>
<td>IPS-2B</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>79.5 (548.2)</td>
</tr>
<tr>
<td>IPS-3A</td>
<td>2.5 Std.</td>
<td>1.5 (38.1)</td>
<td>2.88 (73.1)</td>
<td>0.276 (5.16)</td>
<td>51.0 (351.6)</td>
</tr>
<tr>
<td>IPS-3B</td>
<td></td>
<td></td>
<td></td>
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<td>77.5 (534.4)</td>
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### Table 2-4. Pipe-Pin Details in SFOBB Approach Spans

<table>
<thead>
<tr>
<th>Bent No.</th>
<th>Pin-Pin Detail</th>
<th>Pin Diameter in (mm)</th>
<th>Column Dia. ft (m)</th>
<th>Column Spiral at Top</th>
<th>Longitudinal Reinforcement at Top</th>
<th>Dc/Dp</th>
</tr>
</thead>
<tbody>
<tr>
<td>1U</td>
<td></td>
<td></td>
<td>6.0 (1.829)</td>
<td>2#8@7.5</td>
<td>26#18</td>
<td>5.14</td>
</tr>
<tr>
<td>7U</td>
<td>Design-A</td>
<td>14 (355.6)</td>
<td>5.0 (1.524)</td>
<td>#8@5</td>
<td>16x2#18</td>
<td>4.28</td>
</tr>
<tr>
<td>26U</td>
<td></td>
<td></td>
<td>7.0 (2.133)</td>
<td>#8@5</td>
<td>23x2#18</td>
<td>6.0</td>
</tr>
<tr>
<td>3L</td>
<td></td>
<td></td>
<td>5.0 (1.524)</td>
<td>#8@5</td>
<td>20x2#14</td>
<td>4.28</td>
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<tr>
<td>10L</td>
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<td></td>
<td>5.0 (1.524)</td>
<td>#8@7.5</td>
<td>20x2#14</td>
<td>4.28</td>
</tr>
<tr>
<td>16U</td>
<td>Design-B</td>
<td>20 (508)</td>
<td>7.0 (2.133)</td>
<td>2#8@7.5</td>
<td>28x2#18</td>
<td>4.2</td>
</tr>
<tr>
<td>18U</td>
<td></td>
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<td>8.0 (2.438)</td>
<td>#8@5</td>
<td>33#18</td>
<td>4.8</td>
</tr>
<tr>
<td>22U</td>
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<td></td>
<td>8.0 (2.438)</td>
<td>#8@7.5</td>
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<td>4.8</td>
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Table 2-5. University of Nevada, Reno Shake Table Specifications

<table>
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<tr>
<th>Specification</th>
<th>Value</th>
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<tbody>
<tr>
<td>Table Size</td>
<td>14.0 ft x 14.6 ft (4.3 m x 4.5 m)</td>
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<tr>
<td>Maximum Specimen Mass</td>
<td>100,000 lbm (45,352 kg)</td>
</tr>
<tr>
<td>Maximum Pitch Moment</td>
<td>1,000,000 ft-lb (1,356 kN-m)</td>
</tr>
<tr>
<td>Maximum Yaw Moment</td>
<td>400,000 ft-lb (542 kN-m)</td>
</tr>
<tr>
<td>Maximum Roll Moment</td>
<td>400,000 ft-lb (542 kN-m)</td>
</tr>
<tr>
<td>Force Rating</td>
<td>165,000 lb (734 kN)</td>
</tr>
<tr>
<td>Vertical Live Load</td>
<td>± 150,000 lb (±667 kN)</td>
</tr>
<tr>
<td>Maximum Dynamic Displacement</td>
<td>± 12 inches (±300 mm)</td>
</tr>
<tr>
<td>Maximum Static Displacement</td>
<td>± 14 inches (±350 mm)</td>
</tr>
<tr>
<td>Maximum Velocity</td>
<td>± 40 in/sec (±1000 mm/sec)</td>
</tr>
<tr>
<td>Maximum Acceleration</td>
<td>1 g at 100,000 lbm specimen</td>
</tr>
<tr>
<td>Operating Frequency</td>
<td>0.1-30 Hz</td>
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</table>

Table 2-6. Specification of the Pipe-Pin in the Model and Prototype

<table>
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<tr>
<th>Geometry of</th>
<th>Column Diameter</th>
<th>Steel pipe O.D.</th>
<th>Steel pipe Thickness</th>
<th>Horizontal Gap</th>
<th>Vertical Gap</th>
<th>Can Thickness</th>
<th>Pipe Protrusion</th>
<th>Pipe Embedment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prototype</td>
<td>70 (1778)</td>
<td>14 (355.6)</td>
<td>1.25 (31.75)</td>
<td>1.25 (31.75)</td>
<td>0.75 (19)</td>
<td>0.75 (19)</td>
<td>17.5 (444.5)</td>
<td>65 (1651)</td>
</tr>
<tr>
<td>Model</td>
<td>14 (355.6)</td>
<td>2.88 (73.15)</td>
<td>0.276 (7.0)</td>
<td>0.25 (6.35)</td>
<td>0.15 (3.8)</td>
<td>0.15 (3.8)</td>
<td>3.5 (89)</td>
<td>13 (330)</td>
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</table>
Table 2-7. Concrete Strength for Two-Column Bent Model (PPTC Specimen)

<table>
<thead>
<tr>
<th>Location</th>
<th>Slump before plasticizer in (mm)</th>
<th>Slump after plasticizer in (mm)</th>
<th>Plasticizer volume gal/yd³ (liter/m³)</th>
<th>7 days strength ksi (MPa)</th>
<th>28 days strength ksi (MPa)</th>
<th>Test day strength ksi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Footing</td>
<td>1.125 (28.6)</td>
<td>5.625 (143)</td>
<td>0.25</td>
<td>4.40</td>
<td>6.74</td>
<td>8.15</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(0.723)</td>
<td>(30.32)</td>
<td>(46.48)</td>
<td>(56.16)</td>
</tr>
<tr>
<td>Columns</td>
<td>2 (50.8)</td>
<td>6 (152.4)</td>
<td>0.325</td>
<td>3.49</td>
<td>5.18</td>
<td>6.87</td>
</tr>
<tr>
<td>Cap beam</td>
<td>1.5 (38.1)</td>
<td>3.75 (95.3)</td>
<td>0.3</td>
<td>4.69</td>
<td>7.02</td>
<td>8.07</td>
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</table>

Table 2-8. Specifications for Steel Parts in the Two-Column Bent

<table>
<thead>
<tr>
<th>Element</th>
<th>fy ksi (MPa)</th>
<th>fu ksi (MPa)</th>
<th>Module of elasticity ksi (MPa)</th>
<th>Elongation %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel pipes</td>
<td>52.2 (360)</td>
<td>64 (441)</td>
<td>-</td>
<td>33</td>
</tr>
<tr>
<td>Steel Cans</td>
<td>52.9 (364.7)</td>
<td>61.7 (425.4)</td>
<td>-</td>
<td>31</td>
</tr>
<tr>
<td>Longitudinal bars #4</td>
<td>68.8 (474.4)</td>
<td>108.3 (746.7)</td>
<td>29000</td>
<td>12</td>
</tr>
<tr>
<td>Column spiral W5.5 Φ0.265 in</td>
<td>109 (751.5)</td>
<td>113.6 (783.3)</td>
<td>29000</td>
<td>1.5</td>
</tr>
<tr>
<td>Inner spiral W2.5 Φ0.178 in</td>
<td>55 (379)</td>
<td>71.8 (495)</td>
<td>29000</td>
<td>1.8</td>
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Table 2-9. Mechanical Properties of FRP Tube

<table>
<thead>
<tr>
<th>Property</th>
<th>75° F (24° C) ksi (MPa)</th>
<th>210° F (99° C) ksi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial tensile ultimate stress</td>
<td>10.3 (71)</td>
<td>7.7 (53)</td>
</tr>
<tr>
<td>Axial tensile modules of elasticity</td>
<td>1820 (12548)</td>
<td>1180 (8136)</td>
</tr>
<tr>
<td>Axial compressive ultimate stress</td>
<td>33 (230)</td>
<td>19.4 (134)</td>
</tr>
<tr>
<td>Axial compressive modules of elasticity</td>
<td>1260 (8687)</td>
<td>600 (4137)</td>
</tr>
<tr>
<td>Beam bending ultimate stress</td>
<td>23 (158.6)</td>
<td>16 (110)</td>
</tr>
<tr>
<td>Beam bending modules of elasticity</td>
<td>1460 (10000)</td>
<td>960 (6630)</td>
</tr>
<tr>
<td>Ultimate hoop tensile strength</td>
<td>34 (234)</td>
<td>43.5 (300)</td>
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### Table 3-1. Maximum and Minimum Strains on the Pipe

<table>
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<tr>
<th>Specimen</th>
<th>sg20</th>
<th>sg21</th>
<th>sg22</th>
<th>sg23</th>
<th>sg24</th>
<th>sg25</th>
<th>sg26</th>
<th>sg27</th>
<th>sg28</th>
<th>sg29</th>
<th>sg30</th>
<th>sg31</th>
<th>sg32</th>
<th>sg33</th>
</tr>
</thead>
<tbody>
<tr>
<td>PS1P-A</td>
<td>max</td>
<td>8707</td>
<td>842</td>
<td>580</td>
<td>983</td>
<td>736</td>
<td>410</td>
<td>156</td>
<td>1606</td>
<td>9880</td>
<td>23115</td>
<td>33607</td>
<td>2527</td>
<td>537</td>
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<tr>
<td>PS1P-B</td>
<td>max</td>
<td>488</td>
<td>735</td>
<td>1046</td>
<td>1039</td>
<td>750</td>
<td>417</td>
<td>113</td>
<td>1988</td>
<td>8621</td>
<td>23695</td>
<td>23416</td>
<td>13729</td>
<td>749</td>
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<tr>
<td>PS2P-A</td>
<td>max</td>
<td>538</td>
<td>941</td>
<td>1393</td>
<td>1584</td>
<td>1323</td>
<td>863</td>
<td>205</td>
<td>4187</td>
<td>11266</td>
<td>24833</td>
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<td>-1301</td>
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<td>PS3P-A</td>
<td>max</td>
<td>446</td>
<td>700</td>
<td>799</td>
<td>764</td>
<td>580</td>
<td>283</td>
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<td>36840</td>
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<td>-50</td>
<td>-21</td>
<td>-615</td>
<td>0</td>
<td>-487</td>
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Table 3-2. Equivalent Uniform Bearing Stress Based on Negative Strains

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<tr>
<th>Strain</th>
<th>( M ) kip.in (kN.mm)</th>
<th>( H ) kip (kN)</th>
<th>( f_{c}'/f'_{c} )</th>
<th>( M ) kip.in (kN.mm)</th>
<th>( H ) kip (kN)</th>
<th>( f_{c}'/f'_{c} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>PS1P-A</td>
<td>-2528 187 (21127) 110.025 (489)</td>
<td>4.13</td>
<td>187 (21127) 118.125 (525)</td>
<td>2.75</td>
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</tr>
<tr>
<td></td>
<td>-6814 293 (33103) 120.375 (535)</td>
<td>2.68</td>
<td>293 (33103) 125.1 (556)</td>
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<tr>
<td></td>
<td>-14050 331 (37396) 124.65 (554)</td>
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<tr>
<td></td>
<td>-21240 346.5 (39147) 128.7 (572)</td>
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<td></td>
<td>-28490 355.5 (40164) 132.075 (554)</td>
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<td>355.5 (40164) 133.875 (554)</td>
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<tr>
<td></td>
<td>-35910 358 (40446) -</td>
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<td>358 (40446) -</td>
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<td>-</td>
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<td>-14050 331 (37396) 117.225 (521)</td>
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<tr>
<td></td>
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Table 3-3. Equivalent Uniform Bearing Stress Based on Positive Strains

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<th>sg30, Y = 3.5 in (89 mm)</th>
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<td>H kip (kN)</td>
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PS3P-A | 2023 | 74.92 (8464) | 60.525 (269) | 4.04 | 74.92 (8464) | 63.1125 (280) | 2.29 |
|        | 5959 | 114.4 (12924) | 66.15 (294) | 3.18 | 114.4 (12924) | 68.85 (306) | 2.14 |
|        | 14050 | 134.5 (15195) | 69.75 (310) | 2.83 | 134.5 (15195) | 69.075 (307) | 1.94 |
|        | 21200 | 141.1 (15941) | 68.625 (305) | 2.45 | 141.1 (15941) | 72 (320) | 2.01 |
|        | 28370 | 145.1 (16393) | 70.425 (313) | 2.51 | 145.1 (16393) | 69.525 (309) | 1.84 |
|        | 34700 | 147.6 (16675) | 70.875 (315) | 2.46 | 147.6 (16675) | 69.75 (310) | 1.83 |
|        | Average | 2.91 | 2.01 |

PS3P-B | 2023 | 74.92 (8464) | - | - | 74.92 (8464) | - | - |
|        | 5959 | 114.4 (12924) | - | - | 114.4 (12924) | - | - |
|        | 14050 | 134.5 (15195) | - | - | 134.5 (15195) | - | - |
|        | 21200 | 141.1 (15941) | - | - | 141.1 (15941) | - | - |
|        | 28370 | 145.1 (16393) | - | - | 145.1 (16393) | - | - |
|        | 34700 | 147.6 (16675) | - | - | 147.6 (16675) | - | - |
|        | Average | 2.91 | 2.01 |
Table 3-4. Summary of Bearing Strengths

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<th>Specimen</th>
<th>$D_p$ in (mm)</th>
<th>Compression side strain gauges</th>
<th>Tension side strain gauges</th>
<th>Average $f_c/f'_c$</th>
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</thead>
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<td></td>
<td>$Y_s$ in (mm)</td>
<td>$f'_c/f'_c$</td>
<td>$Y_s$ in (mm)</td>
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<td>2 (51)</td>
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Table 3-5. Comparison of $H_a$ Obtained from the Test and Eq. 3-10

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<th>$H_a$ from test kip (kN)</th>
<th>Average $H_a$ kip (kN)</th>
<th>$f'_c$ Eq. 3-8 ksi (MPa)</th>
<th>$M_p$ Kip.in (kN.mm)</th>
<th>$L_f$ in (mm)</th>
<th>Calculated $H_a$ Eqs. 3-9, 3-10 kip (kN)</th>
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<td></td>
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<td>(578.6)</td>
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<td>(43225)</td>
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<td>(43225)</td>
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Table 3-6. Comparison of Yielding and Ultimate Shear Capacities Obtained from the Test and Formulas

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<th>Equations 3-14, and 3-15</th>
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<td>Ult. Shear kip (kN)</td>
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Table 3-8. Maximum and Minimum Strains in Transverse Spirals in RC Column

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Table 3-9. Maximum and Minimum Longitudinal Strains on the Pipe in RC Column

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Table 3-12. Maximum and Minimum Strains in Longitudinal and Hoop Directions of FRP Tube

| Run | FF11 min | FF11 max | FF13 min | FF13 max | FF21 min | FF21 max | FF23 min | FF23 max | FF31 min | FF31 max | FF32 min | FF32 max | FF33 min | FF33 max | FF34 min | FF34 max | FF41 min | FF41 max | FF42 min | FF42 max | FF43 min | FF43 max | FF44 min | FF44 max | FF51 min | FF51 max | FF52 min | FF52 max | FF53 min | FF53 max | FF54 min | FF54 max |
|-----|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|
| 1   | -20      | 33       | 125      | 290      | 283      | 263      | 53       | 356      | 13       | 257      | 53       | 329      | 20       | 217      | 20       | 237      | 33       |
| 2   | -1252    | 224      | 277      | 1067     | 580      | 2421     | 679      | 3341     | 626      | 2424     | 685      | 2964     | 270      | 1580     | 1213     | 2002     | 699      |
| 3   | -3519    | 152      | 40       | 0        | 250      | 3842     | 679      | 3664     | 942      | 3742     | 705      | 3254     | 422      | 2515     | 1377     | 2272     | 1266     |
| 4   | -10155   | -586     | 0        | 0        | 8020     | 784      | 4521     | 1291     | 9368     | 1390     | 4433     | 2226     | 6394     | 2115     | 3109     | 3237     |
| 5   | 0        | 0        | 0        | 0        | 0        | 1331     | 0        | 2661     | 9520     | 1825     | 0        | 3418     | 9496     | 2438     | 0        | 5189     |
| 6   | 0        | 0        | 0        | 0        | 0        | -3525    | 0        | -3248    | -15073   | -8191    | 0        | -3379    | -6118    | -6287    | 0        | -2842    |
| 7   | 0        | 0        | 0        | 0        | 0        | 1186     | 0        | 415      | 8435     | 3708     | 0        | 9213     | 6139     |

| Run | FF11 min | FF11 max | FF13 min | FF13 max | FF21 min | FF21 max | FF23 min | FF23 max | FF31 min | FF31 max | FF32 min | FF32 max | FF33 min | FF33 max | FF34 min | FF34 max | FF41 min | FF41 max | FF42 min | FF42 max | FF43 min | FF43 max | FF44 min | FF44 max | FF51 min | FF51 max | FF52 min | FF52 max | FF53 min | FF53 max | FF54 min | FF54 max |
|-----|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|
| 2   | -20      | 33       | 125      | 290      | 283      | 263      | 53       | 356      | 13       | 257      | 53       | 329      | 20       | 217      | 20       | 237      | 33       |
| 3   | -1252    | 224      | 277      | 1067     | 580      | 2421     | 679      | 3341     | 626      | 2424     | 685      | 2964     | 270      | 1580     | 1213     | 2002     | 699      |
| 4   | -3519    | 152      | 40       | 0        | 250      | 3842     | 679      | 3664     | 942      | 3742     | 705      | 3254     | 422      | 2515     | 1377     | 2272     | 1266     |
| 5   | -10155   | -586     | 0        | 0        | 8020     | 784      | 4521     | 1291     | 9368     | 1390     | 4433     | 2226     | 6394     | 2115     | 3109     | 3237     |
| 6   | 0        | 0        | 0        | 0        | 0        | 1331     | 0        | 2661     | 9520     | 1825     | 0        | 3418     | 9496     | 2438     | 0        | 5189     |
| 7   | 0        | 0        | 0        | 0        | 0        | -3525    | 0        | -3248    | -15073   | -8191    | 0        | -3379    | -6118    | -6287    | 0        | -2842    |

<p>| Run | FF11 min | FF11 max | FF13 min | FF13 max | FF21 min | FF21 max | FF23 min | FF23 max | FF31 min | FF31 max | FF32 min | FF32 max | FF33 min | FF33 max | FF34 min | FF34 max | FF41 min | FF41 max | FF42 min | FF42 max | FF43 min | FF43 max | FF44 min | FF44 max | FF51 min | FF51 max | FF52 min | FF52 max | FF53 min | FF53 max | FF54 min | FF54 max |
|-----|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|
| 2   | -20      | 33       | 125      | 290      | 283      | 263      | 53       | 356      | 13       | 257      | 53       | 329      | 20       | 217      | 20       | 237      | 33       |
| 3   | -1252    | 224      | 277      | 1067     | 580      | 2421     | 679      | 3341     | 626      | 2424     | 685      | 2964     | 270      | 1580     | 1213     | 2002     | 699      |
| 4   | -3519    | 152      | 40       | 0        | 250      | 3842     | 679      | 3664     | 942      | 3742     | 705      | 3254     | 422      | 2515     | 1377     | 2272     | 1266     |
| 5   | -10155   | -586     | 0        | 0        | 8020     | 784      | 4521     | 1291     | 9368     | 1390     | 4433     | 2226     | 6394     | 2115     | 3109     | 3237     |
| 6   | 0        | 0        | 0        | 0        | 0        | 1331     | 0        | 2661     | 9520     | 1825     | 0        | 3418     | 9496     | 2438     | 0        | 5189     |
| 7   | 0        | 0        | 0        | 0        | 0        | -3525    | 0        | -3248    | -15073   | -8191    | 0        | -3379    | -6118    | -6287    | 0        | -2842    |</p>
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Table 3-13. Maximum and Minimum Longitudinal Strains on the Pipe in FRP tube Column
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Table 4-1. Parameters of the Stick Model

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<th>Pipe O.D. in (mm)</th>
<th>Spacing of the concrete springs in (mm)</th>
<th>Stiffness of the concrete spring kip/in (kN/mm)</th>
<th>Yield strength of concrete springs kip (kN)</th>
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Table 4-2. Concrete Modeling Parameters for FEM (for CDP Material)

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<th>Dilation Angle (deg.)</th>
<th>$f_{fo}/f_{co}$</th>
<th>$\sigma_{co}$ ksi (MPa)</th>
<th>$E$ ksi (MPa)</th>
<th>$\nu$</th>
<th>$\sigma_t$ ksi (MPa)</th>
<th>$\alpha$</th>
<th>$d_1$ in (mm)</th>
<th>$d_2$ in (mm)</th>
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<tbody>
<tr>
<td>Push-off</td>
<td>Main body</td>
<td>6.05 (41.7)</td>
<td>37</td>
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<td>1.16</td>
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<td>3.42 (23.5)</td>
<td>4299 (29641)</td>
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<td>0.0025 (0.063)</td>
<td>0.0179 (0.455)</td>
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<td>2.47 (17.1)</td>
<td>3090 (21308)</td>
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<td>3.76 (25.9)</td>
<td>4753 (32771)</td>
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Table 4-3. Parameters of the Concrete01 Material in OpenSees

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<th>Concrete Material</th>
<th>Compressive Strength kip (kN)</th>
<th>Strain at Maximum Strength</th>
<th>Crushing Strength kip (kN)</th>
<th>Crushing Strain</th>
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<tr>
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<td>-0.55 (-3.79)</td>
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<tr>
<td>Post-Test</td>
<td>Unconfined</td>
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Table 5-1. Parametric Studies Matrix

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<td>Total number of the analyses</td>
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* Bold cases show basic prototype properties.

** Shaded cases indicate that analysis was done for two bearing area diameter:
(1) 3 ft (0.915 m)
(2) Column diameter-1 ft (Column diameter-0.31 m)
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<th>Bearing Area Diameter in (mm)</th>
<th>Parameter Imperial (SI)</th>
<th>Hn Kip (kN) (FEM)</th>
<th>Cracking load Kip (kN) (FEM)</th>
<th>Crack/Ult</th>
<th>Calculated Capacities, Hn From Proposed Design Method, kip (kN)</th>
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<td>Axial Load Index</td>
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<td>60 in (1524 mm)</td>
<td>ALI=0.0</td>
<td>912 (4056)</td>
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<td>914 (4065) 0% 935 (4149) 2%</td>
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<tr>
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<td>ALI=0.06</td>
<td>1523 (6774)</td>
<td>1246 (5542) 0.82</td>
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<td>1526 (6787) 0% 1536 (6832) 1%</td>
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<td>ALI=0.1</td>
<td>1710 (7606)</td>
<td>1371 (6098) 0.80</td>
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<td>1246 (5542) 0.82</td>
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<td>1555 (6916) 0.85</td>
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<td>1571 (6987) 0.81</td>
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<td>1984 (8824) 2% 1989 (8847) 2%</td>
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<td>1580 (7027)</td>
<td>1280 (5693) 0.81</td>
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<td>1649 (7334) 4% 1641 (7299) 4%</td>
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<td>1246 (5542) 0.82</td>
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<td>1526 (6787) 0% 1536 (6832) 1%</td>
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<td>#5@6 (F16@76 mm)</td>
<td>1446 (6431)</td>
<td>1215 (5404) 0.84</td>
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<td>1464 (6511) 1% 1483 (6596) 3%</td>
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<td>1595 (7094)</td>
<td>1340 (5960) 0.84</td>
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<td>1257 (5591) 0.74</td>
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<td>1206 (5364) 0.81</td>
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<td>1718 (7641)</td>
<td>1355 (6027) 0.79</td>
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<td>1730 (7695) 1% 1711 (7610) 0%</td>
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<td>#5@3 (F16@76 mm)</td>
<td>1613 (7174)</td>
<td>1320 (5871) 0.82</td>
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<td>1576 (7010) -2% 1582 (7036) -2%</td>
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<td>1502 (6680)</td>
<td>1305 (5804) 0.87</td>
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<td>#8@10 (F25@254 mm)</td>
<td>1847 (8215)</td>
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<td>1847 (8215) 3% 1824 (8113) 1%</td>
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<td>1357 (6035)</td>
<td>11%</td>
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<td>1391 (6187) 13%</td>
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<td>1890 (8406)</td>
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<td>1847 (8215) 3% 1824 (8113) 1%</td>
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<td>1356 (6031) 5% 1394 (6200) 8%</td>
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<td>60 in (1524 mm)</td>
<td>1462 (6502)</td>
<td>1222 (5435) 0.84</td>
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<td>1448 (6440) -1% 1404 (6244) -4%</td>
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<td>1538 (6841)</td>
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<td>1729 (7690) 12% 1743 (7752) 13%</td>
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<td>1366 (6075) 0.86</td>
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Table 6-2. Nominal Capacities of Pipe-Pin Hinges Used in SFOBB

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<th>Bent No.</th>
<th>Pin-Pin Detail</th>
<th>Column Diameter in (mm)</th>
<th>Column Spiral</th>
<th>Inner Spiral</th>
<th>Capacity Based on Current Method kips (kN)</th>
<th>Detailed Design Method</th>
<th>Simple Design Method</th>
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<td>Basic Capacity kips (kN)</td>
<td>Capacity with ALI=0.06 kips (kN)</td>
<td>Basic Capacity kips (kN)</td>
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<td>Design A</td>
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<td>#8@5</td>
<td>#5@3</td>
<td>1983 (8820)</td>
<td>804 (3576)</td>
<td>1280 (5693)</td>
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<td>7U</td>
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<td>72</td>
<td>#8@5</td>
<td>#5@3</td>
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<td>915 (4069)</td>
<td>1526 (6787)</td>
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<td>84</td>
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<td>2461 (10946)</td>
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<td>#5@3</td>
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<td>#8@5</td>
<td>#5@3</td>
<td>3452 (15354)</td>
<td>1606 (7143)</td>
<td>2585 (11498)</td>
</tr>
<tr>
<td>18U</td>
<td></td>
<td>96</td>
<td>3-#8@7.5</td>
<td>#5@3</td>
<td></td>
<td>1736 (7721)</td>
<td>3032 (13486)</td>
</tr>
<tr>
<td>22U</td>
<td></td>
<td>84</td>
<td>2-#8@7.5</td>
<td>#5@3</td>
<td></td>
<td>1495 (6649)</td>
<td>2410 (10719)</td>
</tr>
</tbody>
</table>
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Record Number NGA1084

Earthquake: Northridge 01 1994 01 17 12:31
Magnitude: 6.69
M0: 1.2162E+26
Mechanism: 2
Hypocenter Latitude: 34.2057 | Longitude: -118.554 | Depth: 17.5 (km)
Fault Rupture Length: 18.0 (km) | Width: 24.0 (km)
Average Fault Displacement: 78.6 (cm)
Fault Name: Northridge Blind Thrust
Slip Rate: 1.50 (mm/yr)

Station: DWP 74 Sylmar - Converter Sta
Latitude: 34.3110 | Longitude: -118.490
Geomatix 1: I | Geomatix 2: H | Geomatix 3: D
Preferred Vs30: 251.20 (m/s) | Alt Vs30:
Instrument location:

Epicentral Distance: 13.11 (km) | Hypocentral Distance: 21.87 (km) | Joyner-Boore Distance: 0.00 (km)
Campbell R Distance: 5.35 (km) | RMS Distance: 12.73 (km) | Closest Distance: 5.35 (km)
Pga: 0.7123 (g)
PGV: 109,3800 (cm/sec)
PGD: 52.3500 (cm)

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Sec. 1
- 0.15" [4 mm] expansion joint filler
- 0.3" [8 mm]
- 10.2" [259 mm]
- 22" [559 mm]

Sec. 2
- 4.5" [114 mm]
- 0.5" [13 mm]
- 4-#3 bars
- Pipe Confining Spiral
  9" [229 mm] φ W3.5 spiral @ 1.5" [38 mm]
- Shear studs are H4L
  Anchors spaced at 1.75" [44 mm] with 1/4" [6 mm] φ and protrude 1" [25.4 mm]

Sec. 3
- Column Spiral
  21-1/2" [537 mm] φ #3 spiral @ 1.5" [38 mm]
- 12-#11 bars
- 18" [457 mm]

Dimensions:
- 30" [762 mm]
- 48" [1219 mm]
- 5" [127 mm]
- 20" [508 mm]
- 67" [1702 mm]
- 70" [1778 mm]
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Fig. 5-45. Crack Width when ALI=1.3, and Friction Coefficient is 0.7, Case (1)
Fig. 5-46. Von Misses Stress when ALI=1.3, and Friction Coefficient is 0.7, Case (1)

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Fig. 5-57. Crack Width when ALI=1.0, and Friction Coefficient is 1.0, Case (1)
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Fig. 5-64. Crack Width when Column Spiral is #8@10 in (254 mm), Case (1)
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Fig. 5-95. Crack Width when Inner Spiral is #5@3 in, Cage Dia.=42 in, Case (1)

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Fig. 5-159. Crack Width when Can Thickness is 0.75 in (19 mm)
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Concrete crushing

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a) Pinned superstructure-column connections

b) Pinned column-footing connection

Fig. 7-14.  Longitudinal Loading

a) Pinned superstructure-column connections

b) Pinned column-footing connection
Fig. 7-15. Uplift Restraining System
APPENDIX A- DESIGN OF THE
PIPE-PIN HINGES FOR TWO-
COLUMN BENT
The proposed design guideline is used to calculate the design capacity of the pipe-pins used in the two-column bent specimen. Columns are 14 in (355.6 mm) diameter with length of 59.5 in (1511.3 mm) and 50 kip (222.4 kN) axial load. The material characteristics and reinforcement details are listed here:

- $f_y : 45 \text{ ksi, (310.3 MPa)}$
- $f_{ys} : 65 \text{ ksi, (448 MPa)}$
- $f_{FRP} : 34 \text{ ksi, (234.4 MPa)}$
- $f'_c : 5 \text{ ksi, (34.5 MPa)}$
- $B : 14 \text{ in, (356 mm)}$
- $D_p : 2.88 \text{ in, (73 mm)}$
- $t_{pipe} : 0.276 \text{ in, (7 mm)}$
- $D_{bearing} : 8.75 \text{ in, (222 mm)}$

Step 1)

1a) $r_1 = 1.44 \text{ in, (36.6 mm)}$

$r_2 = 1.164 \text{ in, (29.6 mm)}$

$Z_{pipe} = \frac{4}{3}(1.44^3 - 1.164^3) = 1.879 \text{ in}^3 (30791 \text{ mm}^3)$

$M_u = 1.1 \times 45 \times 1.879 = 93 \text{ kip.in, (10507 kN.mm)}$

1b) From Detailed method:

For conventional RC concrete column (RC): $L_1 = 3.064 \text{ in, (77.8 mm)}$

For FRP tube column (FRP): $L_1 = 1.92 \text{ in, (48.76 mm)}$

For RC: $f_1 = 4.77 \text{ ksi, (32.89 MPa)}$

For FRP: $f_1 = 9.37 \text{ ksi, (64.6 MPa)}$
1c) For RC: \( H_o = 4.77 \times 3.064 \times 2.88 = 42.13 \text{ kip}, \ (187.4 \text{ kN}) \)

For FRP: \( H_o = 9.37 \times 1.92 \times 2.88 = 51.8 \text{ kip}, \ (230.5 \text{ kN}) \)

1d) \( A_g = \pi(1.44^2 - 1.164^2) = 2.258 \text{ in}^2, \ (1456.7 \text{ mm}^2) \)

\( A_{conc} = \pi(1.164^2) = 4.257 \text{ in}^2, \ (2746 \text{ mm}^2) \)

\[
H_{o,\text{Shear}} = \left\{ \frac{2\sqrt{3}}{3\pi} \left( 45 \times 2.258 + 0.93\sqrt{5} \times 4.257 \right) = 46.2 \text{ kip} \right. \\
\left. \frac{2\sqrt{3}}{3\pi} \left( 310 \times 1456.7 + 2.47\sqrt{34.5} \times 2746 \right) = 318.5 \text{ kN} \right. \\
\]

For RC: \( H_o = 42.13 \text{ kip}, \ (187.4 \text{ kN}) \)

For FRP: \( H_o = 46.2 \text{ kip}, \ (318.5 \text{ kN}) \)

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**Step 2)**

2a) \( \alpha_1 = \cos^{-1}\left( \frac{8.75}{14} \right) = 0.896 \) (rad)

\[
A_c' = \frac{2\pi - 2 \times 0.896 + \sin(2 \times 0.896)}{2} \left( \frac{14}{2} \right)^2 - \frac{\pi \times 2.88^2}{4} = 127.44 \text{ in}^2, \ (82219 \text{ mm}^2) \\
\]

2b) \( f_c' = \begin{cases} 
0.1423\sqrt{5} = 0.318 \text{ ksi} \\
0.374\sqrt{34.5} = 2.2 \text{ MPa}
\end{cases} \)

\( V_c' = 0.8 \times 127.44 \times 5 \cdot \tan(54^\circ) = 44.65 \text{ kip}, \ (28806 \text{ kN}) \)

For RC:

\[
V_{c1}' = 0.5 \times 0.055 \times 65 \times 12.65 \tan(54^\circ) \cdot \left[ \cos(0.896) \cdot \sin(0.896) + \pi - 0.896 \right] \\
= 44.6 \text{ kip} \\
(198.4 \text{ kN})
\]
For FRP:

\[ V'_{s1} = \frac{0.5 \times 0.269 \times 34 \times 14 \times \tan(54^\circ) \cdot \cos(0.896) \cdot \sin(0.896) + \pi - 0.896}{2 \times 1.5} \]

\[ = 120.45 \text{ kip} \]

\( (198.4 \text{ kN}) \)

\[ V'_{s2} = \frac{0.5 \times 0.02 \times 65 \times 8.752 \times \tan(54^\circ) \cdot \pi}{2 \times 0.75} \]

\[ = 13.7 \text{ kip, (60.94 kN)} \]

\[ V_{pipe} = \frac{1.45 \times 93}{8.75 + 2.44} = 11.16 \text{ kip, (49.6 kN)} \]

For RC: \( H_{cr} = 100.82 \text{ kip, (448.4 kN)} \)

For FRP: \( H_{cr} = 100.82 \text{ kip, (190.42 kN)} \)

2c) \( N_u = 127.44 \cdot (1 \text{ ksi}) = 127.44 \text{ kip, (567 kN)} \)

2d) For RC: \( H_a = 42.13 + \left(100.82 - 42.13 \left( \frac{50}{127.44} \right)^0.7 \right) = 72.6 \text{ kip, (322.9 kN)} \)

For FRP: \( H_a = 46.2 + \left(190.42 - 46.2 \left( \frac{50}{127.44} \right)^0.7 \right) = 121.15 \text{ kip, (538.8 kN)} \)

2e) From the moment-curvature analysis:

\( EI = 4.6e6 \text{ kip.in}^2 \) (1.32e10 kN.mm\(^2\))

\[ F_{impact} = 1.9 \cdot 2 \times 0.15 \times 4.6e6 \cdot \frac{59.5^3}{59.5} = 12.45 \text{ kip, (55.38 kN)} \]

For RC: \( H_u = 0.75(72.6) - 12.45 = 42.03 \text{ kip, (186.95 kN)} \)

For FRP: \( H_u = 0.75(121.15) - 12.45 = 78.4 \text{ kip, (348.7 kN)} \)
APPENDIX B- STRAIN RESULTS OF PUSH-OFF SPECIMENS
Fig. B-1. Shear Strain in PS1P-A

Fig. B-2. Shear Strain in PS1P-B
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Fig. B-4. Shear Strain in PS2P-B
Fig. B-5. Shear Strain in PS3P-A

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Fig. B-23. Strains Gauges on Concrete Surface in PS3P-A
Fig. B-24. Strains Gauges on Concrete Surface in PS3P-B
Fig. B-25. Strains Gauges on Spirals in PS1P-A
Fig. B-26. Strains Gauges on Spirals in PS1P-B
Fig. B-27. Strain Gauges on Spirals in PS2P-A
Fig. B-28. Strains Gauges on Spirals in PS2P-B
Strain Gauge 10

Strain Gauge 11

Strain Gauge 12

Strain Gauge 13

Strain Gauge 14

Strain Gauge 15

Strain Gauge 16

Strain Gauge 17

Fig. B-29. Strains Gauges on Spirals in PS3P-A
Fig. B-30. Strains Gauges on Spirals in PS3P-B
APPENDIX C- STRAIN RESULTS OF
TWO-COLUMN PIER MODEL
Fig. C-1. Longitudinal Bar Strain Gauge C11, RC Column

Fig. C-2. Longitudinal Bar Strain Gauge C13, RC Column
Fig. C-3. Longitudinal Bar Strain Gauge C21, RC Column

Fig. C-4. Longitudinal Bar Strain Gauge C22, RC Column
Fig. C-5.   Longitudinal Bar Strain Gauge C23, RC Column

Fig. C-6.   Longitudinal Bar Strain Gauge C24, RC Column
Fig. C-7.  Longitudinal Bar Strain Gauge C31, RC Column

Fig. C-8.  Longitudinal Bar Strain Gauge C32, RC Column
Fig. C-9. Longitudinal Bar Strain Gauge C33, RC Column

Fig. C-10. Longitudinal Bar Strain Gauge C34, RC Column
Fig. C-11. Longitudinal Bar Strain Gauge C41, RC Column

Fig. C-12. Longitudinal Bar Strain Gauge C42, RC Column
Fig. C-13. Longitudinal Bar Strain Gauge C43, RC Column

Fig. C-14. Longitudinal Bar Strain Gauge C44, RC Column
Fig. C-15.  Longitudinal Bar Strain Gauge C51, RC Column

Fig. C-16.  Longitudinal Bar Strain Gauge C53, RC Column
Fig. C-17. Transverse Steel Strain Gauge CS11, RC Column

Fig. C-18. Transverse Steel Strain Gauge CS12, RC Column
Fig. C-19. Transverse Steel Strain Gauge CS13, RC Column

Fig. C-20. Transverse Steel Strain Gauge CS14, RC Column
Fig. C-21.  Transverse Steel Strain Gauge CS21, RC Column

Fig. C-22.  Transverse Steel Strain Gauge CS22, RC Column
Fig. C-23. Transverse Steel Strain Gauge CS23, RC Column

Fig. C-24. Transverse Steel Strain Gauge CS24, RC Column
Fig. C-25. Steel Pipe Longitudinal Strain Gauge CP1, RC Column

Fig. C-26. Steel Pipe Longitudinal Strain Gauge CP2, RC Column
Fig. C-27.  Steel Pipe Longitudinal Strain Gauge CP3, RC Column

Fig. C-28.  Steel Pipe Longitudinal Strain Gauge CP4, RC Column
Fig. C-29.  Steel Pipe Longitudinal Strain Gauge CP5, RC Column

Fig. C-30.  Steel Pipe Longitudinal Strain Gauge CP6, RC Column
Fig. C-31. Steel Pipe Longitudinal Strain Gauge CP7, RC Column

Fig. C-32. Steel Pipe Longitudinal Strain Gauge CP8, RC Column
Fig. C-33. Steel Pipe Shear Strain, West, RC Column

Fig. C-34. Steel Pipe Shear Strain, East, RC Column
Fig. C-35.  Column Spiral in Hinge Area, C111, RC Column

Fig. C-36.  Column Spiral in Hinge Area, C112, RC Column
Fig. C-37. Column Spiral in Hinge Area, C113, RC Column

Fig. C-38. Column Spiral in Hinge Area, C121, RC Column
Fig. C-39.  Column Spiral in Hinge Area, C122, RC Column

Fig. C-40.  Column Spiral in Hinge Area, C123, RC Column
Fig. C-41.  Inner Spiral in Hinge Area, C211, RC Column

Fig. C-42.  Inner Spiral in Hinge Area, C212, RC Column
Fig. C-43. Inner Spiral in Hinge Area, C213, RC Column

Fig. C-44. Inner Spiral in Hinge Area, C221, RC Column
Fig. C-45. Inner Spiral in Hinge Area, C222, RC Column

Fig. C-46. Inner Spiral in Hinge Area, C223, RC Column
Fig. C-47. Longitudinal Bar Strain Gauge F11, FRP Tube Column

Fig. C-48. Longitudinal Bar Strain Gauge F13, FRP Tube Column
Fig. C-49. Longitudinal Bar Strain Gauge F21, FRP Tube Column

Fig. C-50. Longitudinal Bar Strain Gauge F22, FRP Tube Column
Fig. C-51. Longitudinal Bar Strain Gauge F23, FRP Tube Column

Fig. C-52. Longitudinal Bar Strain Gauge F24, FRP Tube Column
Fig. C-53. Longitudinal Bar Strain Gauge F31, FRP Tube Column

Fig. C-54. Longitudinal Bar Strain Gauge F32, FRP Tube Column
Fig. C-55. Longitudinal Bar Strain Gauge F33, FRP Tube Column

Fig. C-56. Longitudinal Bar Strain Gauge F34, FRP Tube Column
Fig. C-57. Longitudinal Bar Strain Gauge F41, FRP Tube Column

Fig. C-58. Longitudinal Bar Strain Gauge F42, FRP Tube Column
Fig. C-59. Longitudinal Bar Strain Gauge F43, FRP Tube Column

Fig. C-60. Longitudinal Bar Strain Gauge F44, FRP Tube Column
Fig. C-61.  Longitudinal Bar Strain Gauge F51, FRP Tube Column

Fig. C-62.  Longitudinal Bar Strain Gauge F53, FRP Tube Column
Fig. C-63.  Longitudinal Strain Gauge on Tube FF11, FRP Tube Column

Fig. C-64.  Longitudinal Strain Gauge on Tube FF13, FRP Tube Column
Fig. C-65.  Longitudinal Strain Gauge on Tube FF21, FRP Tube Column

Fig. C-66.  Longitudinal Strain Gauge on Tube FF23, FRP Tube Column
Fig. C-67. Longitudinal Strain Gauge on Tube FF31, FRP Tube Column

Fig. C-68. Longitudinal Strain Gauge on Tube FF33, FRP Tube Column
Fig. C-69. Longitudinal Strain Gauge on Tube FF41, FRP Tube Column

Fig. C-70. Longitudinal Strain Gauge on Tube FF43, FRP Tube Column
Fig. C-71. Longitudinal Strain Gauge on Tube FF51, FRP Tube Column

Fig. C-72. Longitudinal Strain Gauge on Tube FF53, FRP Tube Column
Fig. C-73.  Hoop Strain on Tube FF32, FRP Tube Column

Fig. C-74.  Hoop Strain on Tube FF34, FRP Tube Column
Fig. C-75.  Hoop Strain on Tube FF42, FRP Tube Column

Fig. C-76.  Hoop Strain on Tube FF44, FRP Tube Column
Fig. C-77.  Hoop Strain on Tube FF52, FRP Tube Column

Fig. C-78.  Hoop Strain on Tube FF54, FRP Tube Column
Fig. C-79. Shear Strain on Tube FF42S, FRP Tube Column

Fig. C-80. Shear Strain on Tube FF44S, FRP Tube Column
Fig. C-81. Shear Strain on Tube FF52S, FRP Tube Column

Fig. C-82. Shear Strain on Tube FF54S, FRP Tube Column
Fig. C-83. Steel Pipe Longitudinal Strain, FP1, FRP Tube Column

Fig. C-84. Steel Pipe Longitudinal Strain, FP2, FRP Tube Column
Fig. C-85. Steel Pipe Longitudinal Strain, FP3, FRP Tube Column

Fig. C-86. Steel Pipe Longitudinal Strain, FP4, FRP Tube Column
Fig. C-87. Steel Pipe Longitudinal Strain, FP5, FRP Tube Column

Fig. C-88. Steel Pipe Longitudinal Strain, FP6, FRP Tube Column
Fig. C-89. Steel Pipe Longitudinal Strain, FP7, FRP Tube Column

Fig. C-90. Steel Pipe Longitudinal Strain, FP8, FRP Tube Column
Fig. C-91.  Steel Pipe Shear Strain, West, FRP Tube Column

Fig. C-92.  Steel Pipe Shear Strain, East, FRP Tube Column
Fig. C-93. Hoop Strain on Tube in Hinge Area, F111, FRP Tube

Fig. C-94. Hoop Strain on Tube in Hinge Area, F112, FRP Tube
Fig. C-95.  Hoop Strain on Tube in Hinge Area, F121, FRP Tube

Fig. C-96.  Hoop Strain on Tube in Hinge Area, F122, FRP Tube
Fig. C-97. Inner Spiral Strain in Hinge Area, F211, FRP Tube

Fig. C-98. Inner Spiral Strain in Hinge Area, F212, FRP Tube
Fig. C-99. Inner Spiral Strain in Hinge Area, F213, FRP Tube

Fig. C-100. Inner Spiral Strain in Hinge Area, F221, FRP Tube
Fig. C-101. Inner Spiral Strain in Hinge Area, F222, FRP Tube

Fig. C-102. Inner Spiral Strain in Hinge Area, F223, FRP Tube
Fig. C-103. Longitudinal Bars in Cap Beam, CB1

Fig. C-104. Longitudinal Bars in Cap Beam, CB2
Fig. C-105.  Longitudinal Bars in Cap Beam, CB3

Fig. C-106.  Longitudinal Bars in Cap Beam, CB4
Fig. C-107.  Longitudinal Bars in Cap Beam, FB1

Fig. C-108.  Longitudinal Bars in Cap Beam, FB2
Fig. C-109. Longitudinal Bars in Cap Beam, FB3

Fig. C-110. Longitudinal Bars in Cap Beam, FB4
Fig. C-111.  Hoop Strain on Steel Can, CC1, RC Column

Fig. C-112.  Hoop Strain on Steel Can, CC2, RC Column
Fig. C-113. Hoop Strain on Steel Can, FC1, RC Column

Fig. C-114. Hoop Strain on Steel Can, FC2, RC Column
APPENDIX D - OPENSEES MODEL OF TWO-COLUMN PIER MODEL
# SET UP ------------------------------------------------------------------------
# units: kip, inch, sec
wipe;  # clear memory of all past model definitions
set dataDir HimeHistory;
file mkdir $dataDir;  # create data directory
model BasicBuilder -ndm 2 -ndf 3;  # Define the model builder, ndm=#dimension, ndf=#dofs
set PI [expr acos(-1.0)];
set sec 1.;  # define basic units

# define GEOMETRY ---------------------------------------------------------------
set LCol 59.5;  # column length
set DCol 14;
set ODtubeCol 14.567;  # Outer diameter of the FRP tube
set DepthOfBent 18;  # Depth of Bent cap section
set WidthOfBent 18;  # Width of Bent cap section
set Span 84;

# calculated parameters
set Weight 50;  # superstructure weight
set PCol $Weight;  # nodal dead-load weight per column
set g 386.4;  # g.
set Mass [expr (2*$PCol+5)/$g];  # nodal mass

# calculated geometry parameters
set ABent [expr $DepthOfBent*$WidthOfBent];  # cross-sectional area of bent cap
set IzBent [expr pow($DepthOfBent,3)*$WidthOfBent/12];  # Bent cap moment of inertia

# nodal coordinates:
# node No X Y
node 1 [expr -1*$Span/2] 0;
node 3 [expr -1*$Span/2] 0;
node 2 [expr +1*$Span/2] 0;
node 4 [expr +1*$Span/2] 0;
node 10 [expr -1*$Span/2] $LCol;
node 20 [expr +1*$Span/2] $LCol;
node 11 [expr -1*$Span/2+1] $LCol;
node 22 [expr +1*$Span/2-1] $LCol;
node 100 0 $LCol;
node 111 0 [expr 6+$LCol];

# Single point constraints -- Boundary Conditions
# node DX DY RZ
fix 3 1 1 1;
fix 4 1 1 1;
mass 111 [expr 0.947*$Mass] 1e-9 0;  # node#, Mx My Mz, Mass=Weight/g, neglect rotational inertia at nodes
mass 10 [expr 0.0015*$Mass] 1e-9 0;
mass 20 [expr 0.0015*$Mass] 1e-9 0;
mass 11 [expr 0.025*$Mass] 1e-9 0;
mass 22 [expr 0.025*$Mass] 1e-9 0;

#equalDOF $NodeTag $cNodeTag $dof1 $dof2 ...
equalDOF 100 111 3;
equalDOF 100 11 3; equalDOF 100 22 3;
equalDOF 11 10 2;
equalDOF 22 20 2;

# MATERIAL parameters  
#nominal concrete compressive strength
set fc    -8.58;  # CONCRETE Compressive Strength, ksi (+Tension, -Compression)
set Ec   [expr 57*sqrt(-$fc*1000)]; # Concrete Elastic Modulus
# confined concrete
set fc1C   -12.49;  # CONFINED concrete (mander model), maximum stress
set eps1C  -4.590E-3;  # strain at maximum stress
set fc2C   -9;  # ultimate stress
set eps2C   -19.5e-3;  # strain at ultimate stress
# unconfined concrete
set fc1U   $fc;   # UNCONFINED concrete (todeschini parabolic model), maximum stress
set eps1U  -0.002;  # strain at maximum strength of unconfined concrete
set fc2U   [expr 0.05*$fc1U]; # ultimate stress
set eps2U  -0.0025;  # strain at ultimate stress
set lambda  0.1;   # ratio between unloading slope at $eps2 and initial slope $Ec
# tensile-strength properties
set ftC   [expr 0.007*sqrt(-$fc*1000)];  # tensile strength +tension
set ftU   [expr 0.007*sqrt(-$fc*1000)];  # tensile strength +tension
set Ets   [expr $ftU/0.002];  # tension softening stiffness
# FRP confined concrete
# Modified stress-strain relationship for concrete confined by FRP
# Simple Model of Saiidi, M., K. Sureshkumar, and C. Pulido (2005)
set Efiber [expr 1850.0];  # tension modulus of FRP fabric
set ffrp [expr 34.0];  # tensile strength of FRP fabric
set t 0.269;    # FRP tube thickness
set fpc [expr -$fc];         # CONCRETE Compressive Strength, ksi
set tj [expr $t];      # Thickness of FRP fabric
set ej [expr 0.5*$ffrp/$Efiber];  # ultimate cfrp strain
set pcf [expr 4*$tj/($ODtubeCol-2*$t)]; # cfrp volumetric ratio
set fpco [expr $fpc+0.003*$pcf*$Efiber]; # concrete stress at start of post yielding branch
set fr [expr 2.0*$Efiber*$ej*$tj/($ODtubeCol-2*$t)]; # confining pressure (stress) at fibers
set eccu [expr $ej/(0.1-0.25*log($fr/$fpc))]; # radial ultimate strain eccu
set fpcu [expr $fpc+3.5*pow($fr,0.7)];    # ultimate concrete stress

# Steel bars
set Fy 74.97;   # STEEL yield stress
set Fu 123;    # modulus of steel
set Es 29000;  # strain-hardening ratio
set Rs 0.03;   # control the transition from elastic to plastic branches
set cR1 0.925;    # control the transition from elastic to plastic branches
set cR2 0.15;    # control the transition from elastic to plastic branches

uniaxialMaterial Concrete01 $IDconcCore $fc1C $eps1C $fc2C $eps2C # $lambda $ftC $Ets; # build core concrete (confined)
uniaxialMaterial Concrete01 $IDconcCover $fc1U $eps1U $fc2U $eps2U # $lambda $ftU $Ets; # build cover concrete (unconfined)
uniaxialMaterial Steel02 $IDreinf $Fy $Es $Bs $R0 $cR1 $cR2;     # build reinforcement material

# Generate a circular reinforced concrete section
# with one layer of steel evenly distributed around the perimeter and a confined core.
# notes
The center of the reinforcing bars are placed at the inner radius
The core concrete ends at the inner radius (same as reinforcing bars)
The reinforcing bars are all the same size
The center of the section is at (0,0) in the local axis system
Zero degrees is along section y-axis

set ri 0.0; # inner radius of the section, only for hollow sections
set ro [expr $DSec/2]; # overall (outer) radius of the section
set nfCoreR 8; # number of radial divisions in the core (number of “rings”)
set nfCoreT 18; # number of theta divisions in the core (number of “wedges”)
set nfCoverR 2; # number of radial divisions in the cover
set nfCoverT 18; # number of theta divisions in the cover

# Define the fiber section 1
section fiberSec $SecTag1 {
    set rc [expr $ro-$coverSec+.5]; # Core radius
    set rb [expr $ro-$coverSec]; # Bars radius
    patch circ $IDconcCore $nfCoreT $nfCoreR $rc $ri 0 360; # Define the core patch
    patch circ $IDconcCover $nfCoverT $rc $ro 0 360; # Define the cover patch
    set theta [expr 360.0/$numBarsSec1]; # Determine angle increment between bars
    layer circ $IDreinf $numBarsSec1 $barAreaSec1 0 0 $rb $theta 360; # Define the reinforcing layer
}

# Define the fiber section 2
section fiberSec $SecTag2 {
    set rc2 [expr $ro2-$t]; # Core radius
    set rb [expr $ro2-$coverSec]; # Bars radius
    patch circ $IDFrpIncasesConc $nfCoreT $nfCoreR $ri $rc2 0 360; # Define the core patch
    patch circ $IDFrpTube $nfFRPT $rc2 $ro2 0 360; # Define the cover patch
    set theta [expr 360.0/$numBarsSec2]; # Determine angle increment between bars
    layer circ $IDreinf $numBarsSec2 $barAreaSec2 0 0 $rb $theta 360; # Define the reinforcing layer
}

# Gap parameters
set TGapMatTag 101
set CGapMatTag 102
set FrictionMatTag 103
set PipeTag 104
set GapParallelTag 105
set GapComplete 106

set TGap 0.15
set CGap -0.15
set FrictionForce 25
set Stiffness 1000
set GStiffness 3000
set PinCapacity 32
set PinPure [expr $PinCapacity-$FrictionForce]

uniaxialMaterial ElasticPPGap $TGapMatTag $GStiffness 500 $TGap;     # Tension Gap properties
uniaxialMaterial ElasticPPGap $CGapMatTag $GStiffness -500 $CGap;     # Compression Gap properties
uniaxialMaterial Steel02 $FrictionMatTag $FrictionForce $Stiffness 0.01 30 .925 .15;  # Friction properties
uniaxialMaterial Steel02 $PipeTag $PinCapacity $Stiffness 0.025 18.5 .925 .15;    uniaxialMaterial Parallel $GapParallelTag $TGapMatTag $CGapMatTag $FrictionMatTag;  # Parallel mat
uniaxialMaterial Series $GapComplete $GapParallelTag $PipeTag;

uniaxialMaterial Hysteretic  $IDBondSlipRC 1642 1.58e-3 2403 0.0237 -1642 -1.58e-3 -2403 -0.0237 1 1 0 0 0.5;
uniaxialMaterial Hysteretic  $IDBondSlipFRP 1400 0.00328 2700 0.0253 -1400 -0.00328 -2700 -0.0253 1 1 0 0 0.5;
uniaxialMaterial Elastic  $IDRigid 9e9;

uniaxialMaterial ElasticPPGap $TGapMatTag $GStiffness 500 $TGap;     # Tension Gap properties
uniaxialMaterial ElasticPPGap $CGapMatTag $GStiffness -500 $CGap;     # Compression Gap properties
uniaxialMaterial Steel02 $FrictionMatTag $FrictionForce $Stiffness 0.01 30 .925 .15;  # Friction properties
uniaxialMaterial Steel02 $PipeTag $PinCapacity $Stiffness 0.025 18.5 .925 .15;    uniaxialMaterial Parallel $GapParallelTag $TGapMatTag $CGapMatTag $FrictionMatTag;  # Parallel mat
uniaxialMaterial Series $GapComplete $GapParallelTag $PipeTag;

# Element parameters ----------------------------------------

# define geometric transformation: performs a linear geometric transformation of beam stiffness and resisting force from the basic system to the global-coordinate system
geomTransf PDelta $ColTransfTag ; # associate a tag to column transformation

set numIntgrPts 5; #Columns
element nonlinearBeamColumn 1 1 10 $numIntgrPts $SecTag1 $ColTransfTag;
element nonlinearBeamColumn 2 2 20 $numIntgrPts $SecTag2 $ColTransfTag;

#Bent
element elasticBeamColumn 3 11 100 $ABent $Ec $IzBent $ColTransfTag;
element elasticBeamColumn 4 100 22 $ABent $Ec $IzBent $ColTransfTag;

element elasticBeamColumn 34 100 111 $ABent $Ec $IzBent $ColTransfTag;

#Gap
uniaxialMaterial Elastic 10 10000;

element truss 5 10 11 1.0 $GapComplete;
element truss 6 20 22 1.0 $GapComplete;

#Bond-Slip
element zeroLength 10 1 3 -mat $IDRigid $IDRigid $IDBondSlipRC -dir 1 2 6;

# Define RECORDERS -------------------------------------------------------------
recorder Node -file $dataDir/node111.out -time -node 111 -dof 1 disp;
recorder Node -file $dataDir/RBase.out -time -node 3 4 -dof 3 reaction;
recorder Node -file $dataDir/Disps.out -time -node 10 20 -dof 1 disp;
recorder Node -file $dataDir/BaseRotation.out -time -node 1 2 -dof 3 disp;
recorder Node -file $dataDir/TopRotation.out -time -node 10 20 -dof 3 disp;
recorder Node -file $dataDir/RBaseAxial.out -time -node 3 4 -dof 2 reaction;

recorder Element -file $dataDir/Gap1F.out -time -ele 5 axialForce;
recorder Element -file $dataDir/Gap1D.out -time -ele 5 deformation;
recorder Element -file $dataDir/Gap2F.out -time -ele 6 axialForce;
recorder Element -file $dataDir/Gap2D.out -time -ele 6 deformation;
recorder Element -file $dataDir/FRPstrain1.out -time -ele 2 section 1 fiber -7 0 $IDFrpTube stressStrain;
recorder Element -file $dataDir/FRPstrain2.out -time -ele 2 section 1 fiber 7 0 $IDFrpTube stressStrain;

recorder display DispShape 10 10 500 500 -wipe
# next three commands define viewing system, all values in global coords
vrp 0 0 0 # point on the view plane in global coord, center of local viewing system
vup 0 1 0 # dirn defining up direction of view plane
vpn 0 0 1 # direction of outward normal to view plane
# next three commands define view, all values in local coord system
prp 0 0 50 # eye location in local coord sys defined by viewing system
viewWindow -60 60 -10 100 # view bounds uMin, uMax, vMin, vMax in local coords
plane 0 15 # distance to front and back clipping planes from eye
projection 0 # projection mode
port -100 100 -100 100 # area of window that will be drawn into
fill 1 # fill mode
display 1 0 2

# define GRAVITY --------------------------------------------------------------
#pattern Plain 1 Linear {
# load 10 0 -$PCol 0
# load 20 0 -$PCol 0
#}

# Gravity-analysis parameters -- load-controlled static analysis
set Tol 1.0e-8; # convergence tolerance for test
contRAINTS Plain; # how it handles boundary conditions
numberer Plain; # renumber dofs to minimize band-width (optimization), if you want to
system BandGeneral; # how to store and solve the system of equations in the analysis
test NormDisplnOr $Tol 8; # determine if convergence has been achieved at the end of an iteration step
algorithm Newton; # use Newton's solution algorithm: updates tangent stiffness at every iteration
set NstepGravity 10; # apply gravity in 10 steps
set DGravity [expr 1./$NstepGravity]; # first load increment;
integrator LoadControl $DGravity; # determine the next time step for an analysis
analysis Static; # define type of analysis static or transient
analyze $NstepGravity; # apply gravity
# ------------------------------------------------- maintain constant gravity loads and reset time to zero
loadConst -time 0.0
puts "Model Built"
# source in procedures
# Uniform Earthquake ground motion (uniform acceleration input at all support nodes)
set GMdirection 1; # ground-motion direction
set GMfile "AppliedMotion" ; # ground-motion filenames
set GMdir EQ # set up ground-motion-analysis parameters
set DtAnalysis [expr 0.0065*$sec]; # time-step Dt for lateral analysis
set TmaxAnalysis [expr 82*$sec]; # maximum duration of ground-motion analysis -- should be 50*$sec
# define DAMPING-------------------------------------------------apply Rayleigh DAMPING from $xDamp
# D=salphaM*M + $betaKcurr*Kcurrent + $betaKcomm*KlastCommit + $beatKinit*$Kinit
set xDamp 0.15; # 2% damping ratio
set lambda [eigen 1]; # eigenvalue mode 1
set omega [expr pow($lambda,0.5)]; # period (sec.)
set Tperiod [expr 2*$PI/$omega]; # period (sec.)
set betaKcurr 0.; # K-proportional damping;
set betaKcomm [expr 2.*$xDamp/($omega)]; # K-prop. damping parameter;
set betaKinit 0.; # initial-stiffness proportional damping
rayleigh $alphaM $betaKcurr $betaKinit $betaKcomm; # RAYLEIGH damping
# perform Dynamic Ground-Motion Analysis
# the following commands are unique to the Uniform Earthquake excitation
set IDloadTag 400; # for uniformSupport excitation
# read a PEER strong motion database file, extracts dt from the header and converts the file
# to the format OpenSees expects for Uniform/multiple-support ground motions
source ReadSMDFile.tcl; # read in procedure Multinition
# Uniform EXCITATION: acceleration input

set inFile $GMdir/$GMfile.AT2
set outFile $GMdir/$GMfile.g3; # set variable holding new filename (PEER files have .at2/dt2 extension)
ReadSMDFile $inFile $outFile dt;  # call procedure to convert the ground-motion file
set GMfatt [expr $g*$GMfact];  # data in input file is in g Unifts -- ACCELERATION TH
set AccelSeries "Series -dt $dt -filePath $outFile -factor $GMfatt"; # time series information
pattern UniformExcitation $IDloadTag $GMdirection -accel $AccelSeries ;  # create Uniform excitation

#------------------------------Axial load history
set ForceFileRC "EQ/Achieved_Axial_RC.txt";
set ForceFileFRP "EQ/Achieved_Axial_FRP.txt";
set ForceFact 1.0;     # Vertical Force scaling factor
pattern Plain 100  "Series -dt $dt -filePath $ForceFileRC -factor $ForceFact" {   load 10 0.0 -1.0 0.0;  }; #end pattern
pattern Plain 200  "Series -dt $dt -filePath $ForceFileFRP -factor $ForceFact" {   load 20 0.0 -1.0 0.0;  }; #end pattern

test NormUnbalance 1.0e-6 25
algorithm KrylovNewton
system ProfileSPD
constraints Transformation
#   gamma beta
integrator Newmark 0.5 0.25
numberer RCM
analysis Transient
set ok 0
set maxNumIter 20;
set tol 1e-6;
set testtype EnergyIncr
set DtAnalysis 0.00125;
set tFinal 80
set tCurrent 0.0
while {$tCurrent < $tFinal && $ok == 0} {

    # original algorithm and time step
    test $testtype $tol $maxNumIter 0;
    set ok [analyze 1 $DtAnalysis]

    # analysis did not converge - reduce time step
    if {$ok != 0} {


puts "$ok != 0"
set ok [analyze 1 [expr $DtAnalysis/10.0]];
}

# analysis did not converge – reduce time step
if {$ok != 0} {
    puts "$ok != 0"
    set ok [analyze 1 [expr $DtAnalysis/40.0]];
}

# analysis did not converge – reduce time step
if {$ok != 0} {
    puts "$ok != 0"
    set ok [analyze 1 [expr $DtAnalysis/200.0]];
}

# analysis did not converge – reduce time step
if {$ok != 0} {
    puts "$ok != 0"
    set ok [analyze 1 [expr $DtAnalysis/1000.0]];
}

# analysis did not converge – reduce time step
if {$ok != 0} {
    puts "$ok != 0"
    set ok [analyze 1 [expr $DtAnalysis/5000.0]];
}

# analysis did not converge – reduce time step
if {$ok != 0} {
    puts "$ok != 0"
    set ok [analyze 1 [expr $DtAnalysis/25000.0]];
}

# analysis did not converge – reduce time step and try Newton w/ initial tangent
if {$ok != 0} {

puts "$ok != 0"
test $testtype $tol 1000 2;
algorithm Newton -initial
set ok [analyze 1 [expr $DtAnalysis/1]]
test $testtype $tol $maxNumIter 2;
}

# analysis did not converge – reduce time step and try Broyden
if {$ok != 0} {
puts "Trying Broyden .."
algorithm Broyden 8
set ok [analyze 1 [expr $DtAnalysis/1]]
}

# analysis did not converge – reduce time step and try Newton w/ line search
if {$ok != 0} {
puts "Trying NewtonWithLineSearch .."
algorithm NewtonLineSearch .8
set ok [analyze 1 [expr $DtAnalysis/1]]
algorithm Newton
}

set tCurrent [getTime]
puts $tCurrent
)
puts "Tperiod is":
puts $Tperiod

set lambda [eigen 1];
puts $lambda
set omega [expr pow($lambda,0.5)];
set TperiodF [expr 2*$PI/$omega];
puts "TperiodF is":
puts $TperiodF

puts "Ground Motion Done. End Time: [getTime]"
puts $fpco;
puts [expr 1*2*$fc/$Ec];
puts $fpcu;
puts $eccu;
APPENDIX E- DESIGN EXAMPLES
Pipe-Pin Column Hinge

The proposed design guideline of Chapter 6 is used to calculate the design capacity of a pipe-pin column hinge. For that, assume a pipe-pin with specifications defined here is going to be used in a 6 ft (1.83 m) diameter column with length of 27 ft (8.23 m) and 1100 kip (4893 kN) axial load. The material characteristics and reinforcement details are list here:

\[
\begin{align*}
    & f_y : 40 \text{ ksi, (276 MPa)} & A_{sp1} : 0.31 \text{ in}^2, (200 \text{ mm}^2) \\
    & f_{ys} : 60 \text{ ksi, (413.7 MPa)} & A_{sp2} : 0.79 \text{ in}^2, (510 \text{ mm}^2) \\
    & f_c' : 4.5 \text{ ksi, (31 MPa)} & s_1 : 3 \text{ in, (76.2 mm)} \\
    & B : 72 \text{ in, (1829 mm)} & s_2 : 5 \text{ in, (127 mm)} \\
    & D_p : 14 \text{ in, (355.6 mm)} & d_1 : 42 \text{ in, (1067 mm)} \\
    & t_{pipe} : 1.25 \text{ in, (31.75 mm)} & d_2 : 67 \text{ in, (1702 mm)} \\
    & D_{bearing} : 50 \text{ in, (1270 mm)} & G : 0.75 \text{ in, (19 mm)}
\end{align*}
\]

Step 1)

1a) \( r_1 = 7 \text{ in, (177.8 mm)} \)

\[ r_2 = 5.75 \text{ in, (146 mm)} \]

\[ Z_{pipe} = \frac{4}{3} \left( r^3 - 5.75^3 \right) = 203.85 \text{ in}^3 (3.34e6 \text{ mm}^3) \]

\[ M_u = 1.1 \times 40 \times 203.85 = 8.97e3 \text{ kip.in, (1.014e9 kN.mm)} \]

1b) From Eqs. 6-9 and 6-12:

\[ f_i = f_c' = 4.5 \text{ ksi, (31 MPa)} \]

\[ L_1 = \sqrt{\frac{1.37 \times 8.97e3}{14 \times 4.5}} = 13.96 \text{ in, (355 mm)} \]
Note that from the detailed method the plastic hinge depth (Eq. 6-21) would be:

\[ L_i = 13.23 \text{ in, (336 mm)} \]

1c) \[ H_o = 4.5 \times 13.96 \times 14 = 880 \text{ kip, (3914 kN)} \]

1d) \[ A_g = \pi (7^2 - 5.75^2) = 50.07 \text{ in}^2, (32303 \text{ mm}^2) \]

\[ A_{conc} = \pi (5.75^2) = 103.87 \text{ in}^2, (67012 \text{ mm}^2) \]

\[
H_{o,Shear} = \begin{cases} 
\frac{2\sqrt{3}}{3\pi} \times 40 \times 50.07 + 0.93 \sqrt{4.5 \times 103.87} = 941 > 880 \text{ kip} \\
\frac{2\sqrt{3}}{3\pi} \times 276 \times 32303 + 2.47 \sqrt{31} \times 67012 = 4.19 \times 10^6 > 3.91 \times 10^6 \text{ N}
\end{cases}
\]

ok.

Step 2)

2a) \[ \alpha_1 = \cos^{-1} \left( \frac{50}{72} \right) = 0.803 \text{ (rad)} \]

\[
A'_c = \frac{2\pi - 2 \times 0.803 + \sin(2 \times 0.803) \left( \frac{72}{2} \right)^2 - \pi \times 14^2}{4} = 3524 \text{ in}^2, (2.27 \times 10^6 \text{ mm}^2)
\]

2b) \[ f'_c = \begin{cases} 
0.1423 \sqrt{4.5} = 0.3018 \text{ ksi} \\
0.374 \sqrt{31} = 2.08 \text{ MPa}
\end{cases} \]

\[ V'_c = 0.8 \times 3524 \times 4.5 \cdot \tan(54^\circ) = 1171 \text{ kip, (8209 kN)} \]

\[ V_{v1}' = \frac{0.5 \times 0.79 \times 60 \times 67 \times \tan(54^\circ) \cdot [\cos(0.803) \cdot \sin(0.803) + \pi - 0.803]}{2 \times 5} = 620 \text{ kip} \]

(2757 kN)

\[ V_{v2}' = \frac{0.5 \times 0.31 \times 60 \times 42 \times \tan(54^\circ) \cdot \pi}{2 \times 3} = 281 \text{ kip, (1250 kN)} \]
\[ V_{\text{pipe}} = \frac{1.45 \times 8.97e^3}{50 + 14} = 203 \text{ kip, (903 kN)} \]

\[ H_{cr} = 1171 + 620 + 281 + 203 = 2277 \text{ kip, (10128 kN)} \]

2c) \[ N_u = 3524 \cdot (1 \text{ksi}) = 3524 \text{ kip, (15675 kN)} \]

2d) \[ H_n = 903 + \left(2277 - 903\right)\left(\frac{1100}{3524}\right)^{0.7} = 1512 \text{ kip, (6725 kN)} \]

2e) From the moment-curvature analysis:

\[ EI = 2.56e9 \text{ kip.in}^2 (7.346e12 \text{ kN.mm}^2) \]

\[ F_{\text{impact}} = 1.9 \cdot \frac{0.75 \times 2.56e9}{324^3} = 107 \text{ kip, (476 kN)} \]

\[ H_u = 0.75 (1512) - 107 = 1027 \text{ kip, (4568 kN)} \]

**Pipe Shear Keys**

The same properties as introduced in pipe-pin column hinge are to be used as a pipe shear key on a massive concrete abutment. The lateral capacity can be calculated as:

\[ f_c^* = \left\{ \begin{array}{l} \frac{\sqrt{4.5}}{2.43} \left(2.95 - \frac{3^{1/4}}{3.35}\right) = 4.5 = 8.76 \text{ ksi} \\ \frac{\sqrt{31}}{6.38} \left(2.95 - \frac{355.6}{9.85}\right) = 31 = 60.35 \text{ MPa} \end{array} \right. \]

\[ H_o = 14 \times 8.76 \left(\sqrt{4.2^2 + \frac{2.2 \times 40 \times 203.85}{14 \times 8.76}} - 4.2\right) = 1055 \text{ kip, (4693 kN)} \]

\[ H_{o,\text{Shear}} = \left\{ \begin{array}{l} \frac{2\sqrt{3}}{3\pi} \times 40 \times 50.07 + 0.93\sqrt{4.5 \times 103.87} = 941 \text{ kip} \\ \frac{2\sqrt{3}}{3\pi} \times 276 \times 32303 + 2.47\sqrt{31 \times 67012} = 4.19e6 \text{ N} \end{array} \right. \]
\[ H_u = 0.75(941 + 0.75 \times 1100) = 1324.5 \text{ kip, (5891 kN)} \]

The capacities calculated for the pipe shear key is 29% larger than the capacity of pipe-pin column hinge because concrete shear failure is not the controlling action anymore.