Executive Summary

Seismic Behavior and Design of Telescopic Pipe-Pin Connections

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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 particularly challenging for concrete structures in seismically active regions.

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 hinges are developed to completely eliminate moment transfer at the top of columns while transferring shear and axial force across the column-superstructure joints. 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). A gap between the steel pipe and the can enables the protruded pipe rotate inside the can. 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 capacity provide by friction may not be reliable because its magnitude may decrease over time due to cyclic thermal movement.

1. Research Objective

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 is only based on pure shear capacity of the section of steel pipe. The primary objective of this study was twofold: (1) to investigate the seismic performance 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 and present it in a format to facilitate its adoption in the Caltrans Seismic Design Criteria (SDC) document. The study was comprised of
comprehensive experimental and analytical studies of pipe-pin connections and their components.

2. Experimental Studies

The experiments were aimed at investigating all the possible modes of pipe-pin failure including those associated with bending of the pipe and pure shear. 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 experimental studies are described in depth in Chapters 2 and 3.

2.1. Push-Off Specimens

One possible failure mode of pipe-pin hinges is the failure of concrete due to bearing pressure from the pipe. Therefore, the concrete bearing strength needs to be known. To obtain data on the bearing strength of concrete against steel pipes, three pairs of 1:3.5 scale push-off specimens were tested at the University of Nevada, Reno (UNR). The specimens were labeled PS1P-A/B, PS2P-A/B, and PS3P-A/B. The test variables were the pipe diameter and the confinement around the pipe. Specimens PS1P-A/B and PS2P-A/B incorporated 3-1/2 xx-Strong (“xx- Strong” stands for double-extra strong pipe) steel pipes 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, included 2-1/2 xx-Strong pipes with 2.88-in (73.1-mm) OD and 0.552-in (14-mm) thickness. In PS1P-A/B and PS3P-A/B inner spirals made with W2.9@1 in (25.4 mm) and W1.7@0.75 (19 mm), respectively, were used, which represented #5@3 in (76.2 mm) in prototype. In PS2P-A/B no inner spirals were used to study the influence of inner spirals on the behavior. Figure 2 shows the details of the push-off specimens.
Fig. 2- Details of Push-Off Specimens

Each specimen was first pulled, then the load was reversed and the loading was continued until failure. Under the pull loading, the pipe pushed the concrete towards the free edge. Typically, damage was initiated by two cracks in the concrete, starting from the edge of the pipe as shown by the arrow. In PS1P-A/B and PS2P-A/B the extent of cracking was less severe because of the strength provided by the inner spirals. In PS2P-A/B the side cracks were larger and two more cracks appeared as shown by white arrow, which confirmed the role of the lateral reinforcement in controlling the cracks. Under push loading, the pipe was pressed towards a large body of concrete, and the behavior of the specimens was completely different. The damage started with flaking of concrete next to the pipe. By increasing the load, the concrete next to the pipe continued to crush and the pipe started to bend (Fig. 3b). The measured force-displacement curves for the push-off specimens are presented in Fig. 4.
For each specimen, strain data on the opposite sides of the plastic hinge were used to determine the moments. The moment and associated lateral load was used to calculate equivalent bearing stresses as is explained in Chapter 3. The average bearing strength for PS1P-A/B was $2.28f'_c$, while for PS3P-A/B the average was $2.35f'_c$. For full size pipes an average bearing strength of $2f'_c$ is recommended, which is also the upper limit on concrete bearing strength specified in the ACI code. The following empirical equation was developed based on the experimental results and adopting the form that was developed by Soroushian [1986] for dowel bars.

$$f'_c = \begin{cases} \frac{\sqrt{f'_c}}{2.4} \left(2.95 - \frac{\frac{D_p}{3.35}}{f'_c} \right) & \text{(ksi)} \\ \frac{\sqrt{f'_c}}{6.4} \left(2.95 - \frac{\frac{D_p}{9.85}}{f'_c} \right) & \text{(MPa)} \end{cases}$$

(1)

**2.2. Pure Shear Specimens**

Three pairs of 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 dimensions 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. Figure 5 shows the shear-deformation results.
According to literature, the effective shear area of the pipe section is $A_v = 2A_g/\pi$.

Adding another term to account for the effect of concrete inside the pipe leads to Eq. 2 and 3 for yield and ultimate shear capacities. The coefficients of the second terms in the equations were based on the measured data for the pure shear specimens.

$$V_{y,\text{infilled}} = \frac{2A_g f_y}{\pi \sqrt{3}} + \left[ \frac{0.47A_c \sqrt{f'_c}}{1.23A_g \sqrt{f'_c}} \right] \text{ksi}$$ (ksi)  

$$V_{u,\text{infilled}} = \frac{2A_g f_u}{\pi \sqrt{3}} + \left[ \frac{0.93A_c \sqrt{f'_c}}{2.47A_g \sqrt{f'_c}} \right] \text{ksi}$$ (MPa)

### 2.3. Two-Column Pier Model

This experiment was designed to evaluate the performance and safety of pipe-pin hinges that were designed using the proposed design method. The details of the study are presented in Chapter 6. A scaled two-column bridge pier was constructed for proof testing. The presence of two columns in the pier provided an opportunity to study two different column details, 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. One of the columns with conventional reinforced concrete detail was aimed at studying the performance of the pipe-pin hinges. The other column was constructed using a concrete filled fiber reinforced polymer (FRP) tube element. Pipe-pin hinges were used to connect the cap beam to the columns. A 2.88-in (73-mm) diameter steel pipe was selected for the top hinges. Detail geometric information is illustrated in Fig. 6. The pipe pin-hinges were designed as capacity protected elements for the expected plastic shear in the columns.

Both columns were 59.5 in (1511.3 mm) long. The conventional concrete column had 14 in (355.6 mm) diameter with longitudinal steel ratio of 2.6%. The plastic shear of this
column was 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. Fibers in this product are aligned in ±55°, which provide strength in the longitudinal and hoop directions. The longitudinal steel ratio in the FRP column was 1.04% that led to comparable shear in both columns at approximately 5% drift. The estimated capacity of the FRP column at 12% drift was 48 kip (213 kN). The axial load on each column was 50 kip (178 kN). The total effective weight of the mass rig was 100 kip (444.8 kN) and matched the total axial load of the columns. Figure 6 shows the details of this pier model.

Upon the review of different records, data from Sylmar converter station obtained during the 1994 Northridge, California earthquake was selected for the input ground motion. The pier model was subjected to a set of seven progressive excitations with acceleration scaling factors of 0.1, 0.4, 0.7, 1.0, 1.3, 1.6, and 1.9. The corresponding target PGAs were 0.091g, 0.364g, 0.637g, 0.91g, 1.183g, 1.44g, and 1.729g for runs 1 to 7, respectively. The maximum measured 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 force is plotted against the slippage in pipe-pin hinges in Fig. 7.

Fig. 6- Two-Column Pier Model

The maximum measured 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 force is plotted against the slippage in pipe-pin hinges in Fig. 7.
The maximum tensile strain of the longitudinal reinforcing bars in RC column was 65800 microstrains, which is approximately 28 times the yield strain.

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 remained 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 Fig. 8a shows. The edge of the RC column was chipped off due to contact between the column and the cap beam as pointed by an arrow in Fig. 8a. This damage was expected because the thickness of the hinge throat was too small to prevent gap closure. Two thin cracks formed on the sides of the pipe, but they 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. 8b. The edges of the bearing areas were ground due to large number of load cycles.
Overall, the pipe-pin hinges were found to meet the performance objective by 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. Analytical Studies

An extensive analytical study of different models was also performed in this study. The analytical studies consisted of three parts. First, a simple nonlinear stick model was developed in SAP2000 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 tests. Next, an elaborate nonlinear finite element (FE) model was constructed using ABAQUS and calibrated versus the experimental results of push-off specimens and PF-1. This model was used to study 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 model was utilized to design the two-column bent specimen and develop the seismic loading protocol.

3.1. Simple Stick Models for Pipe Shear Keys

Figure 9 shows a sketch of the stick model. Three types of uncoupled lumped plastic springs were used in this model: axial, rotational, and shear. The gap was included in the model using gap element.

The analytical results showed that although shear yielding initiated the 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 did not occur.

3.2. Finite Element Modeling
The push-off specimens were first modeled using FE method. The models comprised the following components: concrete body, concrete inside the pipe, steel pipe, reinforcing bars, and spiral/s. Figure 10a shows the details of PS1P specimens. Figure 10b compares the calculated and measured load-displacement curves.

As the second phase of FE studies, PF-1 was analyzed under monotonic loading. Figure 11b 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. Von Misses stresses in Fig. 11a confirm the extensive flexural yielding of the steel pipe at a depth of approximately 1.5 times the pipe diameter. Note that the pipe yielded locally in shear at the concrete surface (the dark region on the pipe-pin in hinge throat area in Fig. 11a).

After achieving confidence in the FE models, the Caltrans pipe-pin “Design A” standard detail was selected as the prototype detail to conduct an extensive parametric study. The parameters were: diameter of hinge throat, friction coefficient and axial load index, column spiral, inner spiral, embedded length of the pipe, protruded pipe length and the gap between pipe and the can, hinge throat thickness and column flexibility, exterior can thickness, pipe thickness and diameter, spiral around the can, studs on the pipe, cyclic
loading, column nonlinearity, shape of the column, and the concrete strength. Details of the parametric analyses and results are presented in Chapter 5 of this report. A summary of the general findings is listed below:

- The lateral load-slip response is approximately linear elastic up to the threshold that lateral load overcomes the horizontal friction resistance at the hinge interface. After the friction is overcome, the column shifts suddenly and the steel pipe impacts the steel can. After this point, the pipe comes in contact with the steel can, and the lateral strength consists of the friction force and resistance of the pipe and the adjacent concrete.
- Partial bearing failure occurs in concrete on the edge of the hinge throat when the column rotates and the axial bearing pressure shifts to the opposite side of the hinge throat.
- The pipe shear and flexural stresses increase as the lateral force increases. At approximately the same time with the beginning of flexural yielding in the 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 at an angle of approximately 45 degrees. This failure plane is numbered as (1) in Fig. 12a and also can be recognized in the FE tensile cracking plot of Fig. 12b.
- 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 causes a diagonal tension crack that spreads through the width of the column. This crack is marked by (2) in Fig. 12a. Figure 12b illustrates that both the aforementioned cracks could potentially form in a pipe-pin connection.

![Fig. 12- Cracking Mechanisms in a Pipe-Pin Subjected to Lateral Load](image)

### 3.3. OpenSees Model for the Two-Column Bent

Modeling the two-column pier specimen using OpenSees served three purposes. First, the pre-test analysis results were used to design the shake table model and to select the ground motion. The second purpose was to develop and verify a macro model for pipe-
pin hinges. And the third was to develop a reliable analytical model to be used for parametric studies of the two-column pier. The analytical model is shown in Fig. 13. A compound element was developed in the present study that was capable of duplicating force-slip behavior of the pipe-pin hinges. The description of this macro model is presented in Chapter 4. Figure 14 presents a comparison of the calculated and measured load-displacement relationships for the bent.

4. Design Method

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

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

with a strength reduction factor of 0.75.

![Analytical Model Diagram](image-url)

Fig. 13- Analytical Model of the Two-Column Pier

![Comparison Graph](image-url)

Fig. 14- Comparison of Analytical and Experimental Responses
In the proposed design method, first the “reference lateral load capacity”, $H_o$, associated to the cracking mechanism (1) (Fig. 12a) is estimated. This capacity is smaller than pure shear capacity of the infilled pipe. 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 actual axial load. The ultimate design capacity accounts for the reduction of the nominal capacity due to the impact resulting from the sudden slippage after the friction force at the connection is exceeded.

4.1. Proposed Design Procedure for Adoption in Caltrans SDC

To facilitate the adoption of the proposed method, the proposed method was written in a format that is consistent with the Caltrans Seismic Design Criteria (SDC) [Caltrans, 2006] format presented in this section. An appropriate place for the material is Sec 7.6.
• Nominal Lateral Load Capacity of Pipe-Pin Hinges

The 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.

\[ \varphi H_n > V_o + F_{impact} \quad \varphi = 0.75 \]  \hspace{1cm} (4)

\[ H_n = H_o + (H_{cr} - H_o) \left( \frac{N}{N_u} \right)^{0.7} \] \hspace{1cm} (5)

\[ F_{impact} = 0.9 \sqrt{\frac{N \times G \times EI}{L_c^3}} \leq 0.5N \quad \text{(Modified)} \] \hspace{1cm} (6)

• Reference Lateral Load Capacity

\[ H_o = 1.17 \sqrt{M_u D_p f_{cy}'} \leq \frac{2A_s f_{as}}{\pi \sqrt{3}} + \left[ 0.93A_p \sqrt{f_c'} (\text{kips}) + 2.47A_p \sqrt{f_c'} (\text{kN}) \right] \] \hspace{1cm} (7)

\[ M_u = 1.45 f_{cy} (r_2^2 - r_1^2) \] \hspace{1cm} (8)

• Upper Limit Lateral Load Capacity

\[ H_{cr} = \begin{cases} \text{Factor 1} \times \left( \frac{0.16A_c \sqrt{f_c'} + A_{y1} f_{ys} d_1}{s_1} + \frac{A_{y2} f_{ys} d_2}{s_2} + \frac{1.45M_u}{D_{bearing} + D_p} \right) (\text{kips}) \\ \text{Factor 1} \times \left( \frac{0.4A_c \sqrt{f_c'} + A_{y1} f_{ys} d_1}{s_1} + \frac{A_{y2} f_{ys} d_2}{s_2} + \frac{1.45M_u}{D_{bearing} + D_p} \right) (\text{kN}) \end{cases} \] \hspace{1cm} (9)

Factor 1 = 0.45 \frac{D_{bearing}}{B} + 0.6 \hspace{1cm} (10)

For the circular column: \( A_c = \frac{\pi}{4} \left( B^2 - D_p^2 \right) \) \hspace{1cm} (11)

For the square column: \( A_c = B^2 - \frac{\pi D_p^2}{4} \) \hspace{1cm} (12)

• Maximum Effective Axial Load

\[ N_u = \begin{cases} \text{Factor 1} \times A_c (\text{kips}) \\ 0.007 \times \text{Factor 1} \times A_c (\text{kN}) \end{cases} \] \hspace{1cm} (13)

4.2. Detailing Recommendations
Based on the parametric studies, the following recommendations are made. The background information for each recommendation is presented in Chapter 5.

- Analytical parametric studies showed that short pipe 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 $4.5D_p$ is recommended for the pipe.
- Excessive protruded length could be detrimental to the hinge behavior due to double curvature bending of the pipe inside the can and the resulting moment at the hinge throat, which is undesirable. In contrast, a short protruded length could compromise the bridge integrity if large uplifting forces occur. A length of $1.2D_p$ for the can ensures a stable and constructible detail without leading to double-curvature bending of the pipe.
- 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 normal size pipe-pins.
- The massive concrete in the superstructure provides sufficient confinement around the can, thereby eliminating the need for supplemental confinement spiral around the can.
- Four to six studs welded to the upper part of the can are recommended to stabilize the can inside the cap beam during construction.
- 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.
- It is understood that the small hinge throat of 1 in (25 mm) might lead to spalling 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.
- Analytical parametric studies showed that increasing the diameter of the inner spiral cage would increase the capacity 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.
- Based on the observed failure mechanism, it is found that the spiral at the top of the column contributes significantly to the hinge 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.
- 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 a protruded length of $1.2D_p$ before the gap between the pipe and the can closes.

Figure 17 shows the detail of a typical pipe-pin that incorporates the above mentioned detailing recommendations.
5. Observations and Conclusions

1. Large relative rotations can be accommodated in pipe-pinion 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-pinion 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. 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.

5. The proof test confirmed that the proposed design guideline is reliable and safe. The measured strains on the steel pipe and inner and column spiral remained well below the yield strains.
6. 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.

7. Examination of the hinges after the test revealed that the pipes were straight, intact, and with no sign of damage. The hinge throats were ground during several cycles of loads. Small dents were seen in the can at the point of contact with the pipes.

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

9. 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.

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

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

12. Using the current design method, which only accounts for the pure shear failure of the steel pipe, would overestimate the capacity of pipe-pin hinges.

13. 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.

14. The column spiral contributes significantly to the capacity of pipe pin connections, and should be designed specifically for the pipe-pin joint.

15. When 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 ensure the integrity of the structure.