Executive Summary Report

Development of A Seismic Design Method for Reinforced Concrete Two-Way Bridge Column Hinges

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ABSTRACT

Two-way hinges are commonly used in bridge columns to eliminate column moment transfer to foundation. Currently the shear capacity of two-way hinges is determined using the shear friction method. When subjected to lateral forces such as earthquake load, hinges are under a combination of axial load, shear as well as moment. The shear transfer mechanism is different from the assumptions in the standard shear friction theory. However, very limited studies are available and no rational code provisions for two-way hinge design exists.

A preliminary hinge design method was developed, refined, and finalized based on shake table tests of large-scale column models. Five 1/3-scale reinforced concrete bridge column specimens with two-way hinge details were tested. The main objective of this study was to investigate the performance of two-way hinges subjected to combine vertical and lateral loads including seismic forces, and to develop a comprehensive and reliable design method for practical application. Several major parameters that may affect the hinge and column performance were included in tests, such as the level of axial load, column aspect ratio, column and hinge steel ratio, and the size of hinges. The test results provided useful information. Based on this information the preliminary design method was evaluated and modified. The data showed that regardless of the level of axial load, size of the hinge, hinge steel ratio, and the column aspect ratio, the shear capacity of two-way hinges is much lower than the shear friction theory estimates. A procedure for two-way hinge design including a rational method to determine the shear capacity of the hinge is proposed and evaluated using the test data.
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Introduction

Reinforced-concrete two-way hinge (pin) detail is frequently used at the base of bridge multi-column bents. Three types of reinforcement may be used in two-way hinges: (1) distributed steel bars, (2) clustered steel bars, and 3) steel pipes. New bridges in Nevada use the first type. The primary benefit of hinges is the reduction of moments on the foundation, thus reducing the cost of the foundation. An additional benefit is the lower cost of repairing a damaged two-way hinge compared to that of a damaged full moment connection. In both orthogonal directions, two-way hinges are intended to prevent a build-up of flexural moments while maintaining shear capacity under severe lateral load such as earthquake load. Strong ground motions cause several cycles of large amplitude load reversals. Hinges must be capable of undergoing large deformations while maintaining their integrity. In addition, hinges must continue to carry the axial compression load of the column. Another general desirable feature for pin connections is the ability to dissipate energy during the inelastic-deformation cycles.

The objectives of this study were to evaluate current practice for design of two-way hinge details in reinforced-concrete bridge columns, to experimentally investigate the seismic performance of columns incorporating such details, and to develop a practical design method for the two-way hinge design.

Current Practice

Codified guidelines for the design of two-way hinge details do not currently exist. As a result, there is considerable variation in the design and detailing of two-way hinges. The
approach that is used includes the determination of the size of the hinge based on the columns axial force, and to design for shear across the hinge by providing the amount of longitudinal steel required using the shear friction theory. Except for the studies of Lim, et al. (1991) and Haroun, et al. (1993) very little information on the lateral load response of a two-way hinge has been reported in the literature. Lim, et al.’s study proposed a design procedure based on their test results. The failure mechanism of the column with two-way hinges was considered either flexural failure or shear friction failure at the hinges. The study by Haroun, et al. concluded that the shear failure mechanism of the column with two-way hinge is diagonal shear failure enclosing the whole column section; the strength of the section can be predicted by code specification for reinforced concrete beam shear design.

Many past tests at UNR and the studies discussed above have shown that one-way and two-way hinges experience large rotations under lateral loads and that the traditional shear friction mechanism in which (1) two concrete surfaces are assumed to slip relative to each other and (2) all the bars crossing the interface yield in tension does not exist in two-way hinges.

**Experimental Program**

The specimens used in this study will be referred to as THD-1 to THD-5. In this notation, “T” stands for test specimen, “H” for hinge, “D” for detail, and “1” to “5” to indicate the specimen number. Five, one-third scale specimens were constructed and tested on a shake-table at UNR structures lab using the 1994 Northridge earthquake record obtained at the Sylmar County Hospital ground floor. The Sylmar record was selected as the input
motion for the shake table tests in order to maximize the displacement ductility demand on the specimens without exceeding the limits of the shake table. The parameters and vertical load used in the tests were chosen such that a wide range of realistic conditions for two-way hinges could be simulated and studied.

Major parameters that might affect the hinge and column performance were included in tests. These parameters included the axial load, column aspect ratio, hinge steel ratio, and the diameter of the hinge. Figure 1 shows the typical dimensions and reinforcing details of the test specimens. Table 1 lists the main parameters for the five specimens. All specimens except for THD-3 were designed to fail in shear at the hinge after plastic hinging of the base. Model THD-3 was designed with a relatively large hinge shear capacity so that it would fail in flexure. Columns THD-1 and 2 had different aspect ratios to determine the effect of this parameter on shear strength. In THD-4, the axial load was reduced to near zero to determine the performance of hinges in which the overturning moments have cancelled the axial compression due to dead load. The hinge diameter in THD-5 was reduced to 8 inch to determine the effect of higher axial loads and smaller hinges on the shear capacity. In all specimens, the hinge gap was 1 inch. This would correspond to 3 inch in a full-scale hinge, which is considerably larger than the hinge gaps in typical bridge construction. The gap was intentionally over-sized to avoid gap closure and buildup of large moments at hinges.

The measured concrete compressive strength for the hinges on the testing day was 7.72 ksi, 5.91 ksi, 5.66 ksi, 5.64 ksi, and 5.31 ksi from THD1 to THD5, respectively. The maximum aggregative size for concrete mix in the test specimens was 3/8 inch. The average measured yield strength of the longitudinal hinge rebar was 81 ksi.
All specimens were tested in double curvature mode using one of the MTS (Mechanical Testing & Simulation Systems Corporation) shake tables at University of Nevada, Reno (Figure 2). The axial load system consisted of a steel spreader beam, two high strength steel threaded rods and two hydraulic rams. The inertia mass system was used to apply the lateral inertia force to the column. The mass rig is a pinned frame with concrete blocks placed on its deck.

The Sylmar record was selected as input motion for the shake table tests based on the maximum displacement ductility demands. Time compression factor was applied to the shake-table input motion to take into account of the scale factor of the specimens. The typical test sequence for each specimen is shown in Figure 3. The testing sequence was defined based on the dynamic response obtained from computer simulation program RCShake with the estimated properties of each specimen. Small increments of the Sylmar record were applied to the specimens in order to determine the elastic response. Once the effective yield was reached, the amplitude of the input record was increased until failure.

More than 110 channels of data were collected using strain gages and other instruments placed on the test specimens in order to monitor the behavior at critical sections.

**Test Results**

**Specimen THD-1**
Under 0.5 x Sylmar (peak ground acceleration, PGA=0.3g) motion, visible flexural cracks were formed at the base of the column. At 1.25 x Sylmar (PGA=0.75g), the first
shear crack appeared at the hinge and concrete started to spall at base of the column. Hinge concrete started spalling at 1.5 x Sylmar (PGA=0.9g) together with exposure of the column spirals at the base. Right after the motion of 2.25 x Sylmar (PGA=1.35g), a crack was formed at bottom of loading head and extended to where hinge and header meets, the hinge region deteriorated extensively, with spalling and permanent sliding and rocking behavior. The deterioration increased as the input motion increased. THD-1 failed at 2.875 x Sylmar (PGA=1.725g). The hinge and a small part of the column edge shear off from the top of the column. THD-1 reached the maximum lateral force of 85 kips and displacement of 4.84 inches at top of the column. Figure 4 shows the THD-1 specimen force displacement hysteretic response for THD-1. It shows that the stiffness degraded and the permanent deformation increased while the specimen went through the events. The lateral strength of THD-1 followed classic “back-bone” shape with positive slope until failure occurred.

**Specimen THD-2**

Visible flexural cracks were seen at the base of the column at 0.25 x Sylmar (PGA=0.15g) event. At 0.75 x Sylmar (PGA=0.45g), first hinge crack was formed together with yielding of the longitudinal rebar at column base. Under 1.25 x Sylmar (PGA=0.75g), spalling took place at both the hinge and column base. After the motion of 2.0 x Sylmar (PGA=1.2g), a wide crack was formed at the bottom of the header and extended to the intersection of the hinge and header, along with extensive spalling at the two-way hinge. At 2.25 x Sylmar (PGA=1.35g), hinge gap closed due to the significant hinge rotation. The header hit the edge of the column top and caused minor damage. At
2.5 x Sylmar (PGA=1.5g), combined with the exposure of the hinge spiral, hinge gap closed again. Based on the observation and supplemental studies after the test, it was believed that right after this event dowel action took place at the hinge and started to provide the lateral strength of the hinge. The deterioration substantially increased at the event of 2.625 x Sylmar (PGA=1.575g) as hinge gap was reduced due to the crushing of the hinge concrete. The test was stopped at 3.0 x Sylmar (PGA=1.8g) while the specimen experienced large permanent offset between the header and the column. THD-2 reached a maximum lateral force of 56.2 kips and maximum displacement of 7.34 inches at column top. Figure 5 shows the THD-2 force-displacement hysteretic response for all events. It shows the stiffness degradation, as well as the significant shifting of the hysteresis loops, which indicates the large permanent displacement of the specimen.

**Specimen THD-3**

This specimen was designed to fail in flexure and not hinge shear. At 0.25 x Sylmar (PGA=0.15g), visible flexural cracks appeared at the base of the column. First hinge crack was seen at motion of 0.75 x Sylmar (PGA=0.45g). Under 1.25 x Sylmar (PGA=0.75g), spalling was seen at both hinge region and column base, while cracks were found at the bottom of the header close to the hinge-header joint. At 1.75 x Sylmar (PGA=1.05g), due to the severe spalling of the clear cover at the column base, the longitudinal reinforcement of column was exposed. Hinge gap closed at 2.0 x Sylmar (PGA=1.2g) due to significant rotation at hinge. At 2.25 x Sylmar (PGA=1.35g), hinge gap was reduced due to crushing of concrete by the axial load. THD-3 failed at motion of 2.5 x Sylmar (PGA=1.5g) in flexure at both ends of the specimen. Four longitudinal bars
ruptured at the base, and at least one longitudinal top hinge bar ruptured (because the hinge gap is set back not all the longitudinal hinge bars could be observed). THD-3 reached a maximum lateral force of 51 kips and displacement of 4.42 inches at the top. Figure 6 shows the THD-3 force-displacement hysteretic response. The curves show that the stiffness and strength degraded as the acceleration amplitude increased. The shifting of the hysteresis loops indicates permanent displacement of the specimen.

**Specimen THD-4**

The nominal axial load in this specimen was zero to simulate the effect of uplift due to overturning moments. The first crack was seen at 0.1 x Sylmar (PGA=0.06g). At 0.5 x Sylmar (PGA=0.3g), a diagonal shear crack appeared at the hinge. Concrete spalled in the hinge under 0.75 x Sylmar (PGA=0.45g). THD-4 failed at 1.25 x Sylmar (PGA=0.75g). The hinge longitudinal reinforcement fractured and the header was sheared off at the hinge and separated from column. The specimen reached a maximum lateral force capacity of 38.1 kips, and displacement of 2.94 inches at top. Figure 7 presents the force-displacement hysteretic response for THD-4. The curves show that the stiffness and strength degraded as the acceleration amplitudes increased. Some permanent displacement is also evident. Compared to other specimens, the shear and displacement capacities are significantly lower, mainly due to the zero axial load effect.

**Specimen THD-5**

In this specimen, the hinge diameter was relatively small (50% of the column diameter as opposed to 62.5% in the others). Flexural cracks were visible at the base of the column at
0.25 x Sylmar (PGA=0.15g). At 0.5 x Sylmar (PGA=0.3g), first hinge spalling was seen while the crack was developed at the base of the column. Spalling took place at base of the column at 1.5 x Sylmar (PGA=0.9g). Under motion of 1.75 x Sylmar (PGA=1.05g), hinge gap closed due to the large rotation at the hinge, together with the exposure of the spirals at hinge. Column header showed some twisting. At 2.0 x Sylmar (PGA=1.2g), more significant movement was seen with smaller hinge gap due to the crushing of the hinge concrete. Based on the observation and follow up analysis, it is believed that right after this event, the dowel action took place at the shear plane of hinge. The deterioration under 2.75 x Sylmar (PGA=1.65g) was quite substantial. The permanent deformation was large and the only connection between the header and column was the hinge longitudinal rebar. The test was stopped at 3.0 x Sylmar (PGA=1.8g) while the specimen experienced large permanent deformation and at least one of the longitudinal rebar ruptured at the column base. THD-5 reached a maximum lateral force of 82.9 kips and displacement of 7.57 inches at the top. Figure 8 presents the THD-5 force displacement hysteretic response for all events. Stiffness degradation and significant shifting of the hysteresis loops, which indicate the large permanent displacement of the specimen, can be seen. There was 8% drop of the peak lateral strength during events 2.0 x Sylmar before the dowel action took over under 2.25 x Sylmar.

**Idealized Force-Displacement Curves and Displacement Ductilities**

The envelopes of the measured force displacement curves were idealized by bilinear relationships to quantify the ductility capacity of the specimens. Failure was assumed either by the peak displacement with the corresponding force or 80% of the maximum
force with the corresponding displacement when the force at the peak displacement dropped more than 20% of the maximum force at failure. Figure 9 shows a comparison of the idealized force-displacement plots. The achieved displacement ductility ratio for THD-1 to THD-5 was 6.4, 8.5, 8.1, 6.9, and 14.5, respectively.

Shear Transfer Mechanism in Hinges

The test data were used to establish peak shear capacity and the corresponding interface slip for each specimen. Figure 10 shows envelops of lateral force vs. shear slippage at the hinge for all specimens. Peak shear capacity was defined as the maximum shear force resisted by the specimen.

The shear-friction method has been widely applied in hinge design due to the absence of a more appropriate method. The theory assumes that as the concrete segments on the two sides of an initially cracked section slide relative to each other, they introduce tension in the steel crossing the crack because of the roughness of aggregates. Bridge designers have used the shear friction method to calculate the lateral load strength of hinged column, assuming that all the bars at the hinge yield in tension and that aggregate interlock takes place over the entire hinge section. In reality, lateral forces are introduced to bridge columns through loads acting primarily at the deck level; therefore, a substantial flexural moment is expected at the hinge, thus limiting the contact area to the compression zone of the hinge section. To realistically estimate the hinge shear strength, the effects of flexure (and axial load) must be considered. A rational method to determine the shear capacity of the hinge is proposed in this study in which only the shear strength provided in the contact area is accounted for. The mechanism is shown in Figure
11. The compression zone of the section under flexural and axial load can be determined from standard moment-curvature analyses. By considering the force equilibrium in the section, the compression force in the contact area can be found. The shear capacity of the hinge is the product of this compression force and a proper friction coefficient. The friction coefficient was determined from the experimental results obtained in this study and was verified using limited data available from two-column bent tests conducted in previous studies. Table 2 shows that the measured coefficients ranged from 0.46 to 0.54. Specimen THD-3 was not included because that specimen failed in flexure. A conservative coefficient of 0.45 is recommended.

The test data from this study showed that the conventional shear friction method significantly over-estimates hinge strength by 185% to 250%. The results from the proposed design method led to the results which were conservative and close to the measured data (Table 3).

**Proposed Design Method and Numerical Example**

**Design Method**

On the basis of this investigation and a survey of the literature, the following design recommendations are proposed for the design of two-way hinges.

1) Determine the hinge section and required longitudinal steel:

- Hinge area:

\[ A_g \geq \frac{P}{0.2f'_c} \]

Where:

\[ A_g \]: gross area of hinge section.
P: column axial load.

f'c: concrete compressive strength.

- Use minimum longitudinal reinforcement permitted by AASHTO provision for columns:

\[ A_s \geq 0.01A_g \]

Where:

\( A_s \): hinge longitudinal reinforcement steel area.

2) Perform hinge transverse reinforcement design, using Mortensen and Saiidi’s performance based design method (Reference 6), for a target curvature ductility of 10.

3) Find hinge confined concrete properties. Hinges experience a “double confinement” from the hinge spiral and the confinement provide by the surrounding column above the footing below. Determine the hinge confined concrete properties by incorporating the following modified confined lateral pressure and spiral steel ratio into Mander’s method for \( f'_{cc} \), \( \varepsilon_{cc} \) and \( \varepsilon_{cu} \).

\[
f'_l = \left[ \left( 2f'_yA_{sh}/D's_h \right)_{hinge} + \left( 2f'_yA_{sh}/D's_h \right)_{column} \right]
\]

\[
\rho_s = \left[ \left( 4A_{sh}/D's_h \right)_{hinge} + \left( 4A_{sh}/D's_h \right)_{column} \right]
\]

Where:

\( f'_l \): effective hinge confined lateral pressure.

\( \rho_s \): hinge effective volumetric ratio of confining steel.

\( f'_y \): yield strength of the reinforcement.

\( A_{sh} \): spiral bar area.

\( D' \): diameter of the confined core.

\( s_h \): spiral pitch.
4) Determine the flexural capacity of the hinge using the confined concrete properties and make sure it can be resisted by the footing. Adjust the hinge size, the longitudinal bar ratio, or both as necessary.

5) Calculate hinge shear capacity:
   - Run moment curvature analysis; find section compression force (axial load change due to lateral overturning needs to be included in the analysis):
     \[ C = C_c + C_s = P + T_s \]
   Where:
   - \( C \): result compression force.
   - \( C_c \): compression force at concrete.
   - \( C_s \): compression force in rebar.
   - \( P \): axial load.
   - \( T_s \): tension force in rebar.
   - \( \phi V_n = \phi \mu C = \phi \mu (C_c + C_s) \), use \( \mu = 0.45 \).
   Where:
   - \( \phi \): strength reduction factor, 0.85.

6) Calculate hinge plastic shear demand:
   \[ V_u = (M_c + M_h) / L \]

7) Check to see if \( \phi V_n > V_u \), if not adjust either the hinge longitudinal bars, the size of hinge section, or both and repeat steps 1 to 7 until the shear capacity is sufficient.

8) Check for hinge gap closure. Determine if \( \theta_n < \theta_{close} \).
   - Assume a hinge gap, \( g = 100 \text{ mm} \) (4 in).
• Two-way hinge ultimate rotation capacity:

\[ L_p = g + 0.15f_yd_h \]  \hspace{1cm} (f_y \text{ in ksi})

\[ \theta_n = \theta_s + \theta_p \]

\[ \theta_c = g \phi_y \]

\[ \theta_p = L_p (\phi_u - \phi_y) \]

• Two-way hinge rotation for hinge closure:

\[ \theta_{\text{close}} = \sin^{-1}\left(\frac{g}{0.5D}\right) \]

Where:

D: column diameter.

• Check to see if \( \theta_n < \theta_{\text{closure}} \), if not, increase hinge gap until it is sufficient to prevent closure.

9) Detailing of the two-way hinge section:

• Distribute the hinge longitudinal reinforcement around the section.

• Provide spiral for the hinge section, and extend into column and footing at least a distance of \( 1.25L_d \) (\( L_d = \) longitudinal bar tension development length).

**Numerical Example:**

A circular bridge column has a clear height of 20 ft and diameter of 60 inch, 3% of longitudinal steel ratio, with column spiral of #7@4 inch. Column design axial load and plastic moment are \( P_u = 0.08A_gf'c = 1131 \) kips and \( M_u = 11875 \) kip-ft. The concrete compressive strength is 5000 psi, concrete strain at peak strength, \( \varepsilon_c \), is 0.0021, the
concrete ultimate strain, \( \varepsilon_u \), is 0.0035, the steel yield strength, \( f_y \), is 60 ksi, steel ultimate strength, \( f_u \), is 80 ksi, steel strain at beginning of strain hardening, \( \varepsilon_{sh} \), is 0.015, and steel strain at ultimate strength, \( \varepsilon_{su} \), is 0.15.

1) Determine the hinge area and hinge longitudinal steel:

\[
A_g = \frac{P}{0.2f'_c} = \frac{1131}{(0.2 \times 5)} = 1131 \text{ in}^2
\]

Based on the required area, hinge diameter is:

\[
\phi = 38 \text{ inch}, \text{ with } A_g = 1134 \text{ in}^2.
\]

\[
A_{s \text{ min}} = 0.01 \times 1134 = 11.34 \text{ in}^2
\]

Use 20-#7, \( A_s = 12 \text{ in}^2 > 11.34 \text{ in}^2 \) O.K.

1st trial will be the section with \( \phi = 38 \) inches and 20-#7 longitudinal steel!

2) Using the Mortensen and Saiidi method (Reference 6) design hinge spiral:

- Run M-\( \phi \) analysis of the section which was defined at step 1, using unconfined concrete properties for the section. Find \( c \) and \( \phi_y \) and calculate \( \phi_u \) and \( \varepsilon_{cu} \) with target curvature ductility of \( \mu = 10 \):

\[
c = 13.54 \text{ inches}
\]

\[
\phi_y = 0.0001094 \text{ rad/in}
\]

\[
\phi_u = \mu \phi_y = 10 \times 0.0001094 = 0.001094 \text{ rad/in}
\]

\[
\varepsilon_{cu} = c \phi_u = 13.54 \times 0.001094 = 0.0148
\]

- Calculate spiral steel ratio:

\[
\rho_{sp} = \left( \varepsilon_{cu} - 0.004 \right) \left( f'_c / f_y \varepsilon_{su} \right) = (0.0148 - 0.004)(5/60/0.15) = 0.006
\]

Use #3 @ 2” spiral, \( \rho_{sp} = 4A_s / D's = 4 \times 0.11 / (33 \times 2) \)

\[
= 0.0067 > 0.006 \text{ O.K!}
\]
3) Determine hinge concrete confinement properties using Mander’s equation with modified confined lateral pressure and spiral steel ratio for hinge core concrete:

\[
f_l = \left[2f_y A_{sh} / d_{sh} s_{h} \right]_{hinge} + \left[2f_y A_{sh} / d_{sh} s_{h} \right]_{column}
\]

\[
= 2 \times 60 \times 0.11 / (33 \times 2) + 2 \times 60 \times 0.6 / (55.1 \times 4)
\]

\[
= 0.5265 \text{ ksi}
\]

\[
\rho_s = \left[\left(4A_s / D_s\right)_{column} + \left(4A_s / D_s\right)_{hinge}\right]
\]

\[
= 4 \times 0.6 / (55.1 \times 4) + 4 \times 0.11 / (33 \times 2)
\]

\[
= 0.0176
\]

\[
f'_{cc} = \left(-1.254 + 2.254 \sqrt{1 + 7.94 f'_y / f'_c - 2 f'_y / f'_c} \right) f'_c
\]

\[
= (-1.254 + 2.254 \times (1+7.94 \times 0.5265 / 5 )^{1/2}-2 \times 0.5265 / 5 ) \times 5
\]

\[
= 7.948 \text{ ksi}
\]

\[
\varepsilon_{cc} = 0.002 \left[1 + 5 \left(f'_y / f'_c - 1 \right) \right]
\]

\[
= 0.002 \times (1+5 (7.948 / 5-1))
\]

\[
= 0.0079
\]

\[
\varepsilon_{cu} = 0.004 + 1.4 \rho_s f_y \varepsilon_{yu} / f'_c
\]

\[
= 0.004 + 1.4 \times 0.0176 \times 60 \times 0.15 / 7.948
\]

\[
= 0.0319
\]

For the concrete at clear cover of the hinge, use column confined core properties which are based on Mander’s model, \( f'_{cc} = 6.965 \text{ ksi}, \varepsilon'_{cc} = 0.0059, \varepsilon'_{cu} = 0.0237. \)

4) Check if the footing is sufficiently strong to resist the hinge moment. From M-\( \phi \) analysis of the hinge using confined concrete properties:

\[ M_{hu} = 2208 \text{ kip-ft} = 18.6\% \text{ of column plastic moment.} \]
Assume OK!

5) Find hinge shear capacity, based on results of new M-\(\phi\) analysis:

\[ M_{hu} = 2208 \text{ kip-ft.} \]

\[ T_s = 561.3 \text{ kips} \]

\[ C = T_s + P = 561.3 + 1131 = 1692.3 \]

\[ \phi V_n = \phi \mu C = 0.85 \times 0.45 \times 1692.3 = 647.3 \text{ kips} \]

6) Calculate column plastic shear demand based on column and hinge section plastic moments.

\[ V_u = \frac{M_u + M_{hu}}{L} = \frac{11875 + 2208}{20} = 704.2 \text{ kips} \]

7) Check.

\[ V_n = 647.3 \text{ kips} < V_u = 704.2 \text{ kips} \quad \text{Not Good!} \]

Design needs to be revised by increasing either the hinge section area or hinge steel ratio. In this case the difference between the shear demand and capacity is small. Therefore the hinge steel ratio will be increased. Repeat steps 4 to 7 until the design converges.

Repeat step 4: Use 22-#8 as hinge longitudinal steel, re-run M-\(\phi\) analysis.

\[ M_{hu} = 2562.5 \text{ kip-ft} = 21.5\% \text{ of column plastic moment} \]

Assume OK!

Repeat step 5: From the latest M-\(\phi\) analysis, \(T_s = 826.1 \text{ kips.} \)

\[ C = 826.1 + 1131 = 1957 \text{ kips} \]

\[ \phi V_n = 0.85 \times 0.45 \times 1957 = 748.6 \text{ kips} \]

Repeat step 6: New column plastic shear.

\[ V_u = \frac{11875 + 2562.5}{20} = 721.9 \text{ kips} \]

Repeat step 7: \( V_n = 748.6 \text{ kips} > V_u = 721.9 \text{ kips} \)
Design O.K.!

8) Check two-way hinge gap closure, and determine hinge gap thickness:

Assume \( g = 4 \) inch.

\[
L_p = g + 0.15 f_y d_h
\]

\[
= 4 + 0.15 \times 60 \times 1 = 13 \text{ in}
\]

\( \phi_y = 0.00015 \text{ rad/in} \)

\( \phi_u = 0.00396 \text{ rad/in} \)

\[
\theta_c = g \phi_y
\]

\[
= 4 \times 0.00015 = 0.0006 \text{ rad}
\]

\[
\theta_p = L_p (\phi_u - \phi_y)
\]

\[
= 13 \times (0.00396 - 0.00015) = 0.0496 \text{ rad}
\]

\[
\theta_n = \theta_c + \theta_p
\]

\[
= 0.0006 + 0.0496 = 0.0502 \text{ rad}
\]

\[
\theta_{\text{close}} = \sin^{-1}
\left(\frac{g}{0.5D}\right)
\]

\[
= \sin^{-1} \left[\frac{4}{(0.5 \times 60)}\right] = 0.1337 \text{ rad}
\]

\( \theta_n = 0.0502 \text{ rad} < \theta_{\text{closure}} = 0.1337 \text{ rad} \)

Design O.K.

9) Horizontal gap is selected as 4 inch, as recommended.

Figure 12 shows the structural detail cross section for the two-way hinge of the design example problem.
Conclusions

Based on the tests and analyses reported in this article, the following observations and conclusions can be made.

1) The shear friction theory significantly over-estimated the shear strength; the shear failure mechanism of the hinge region was very different from that assumed in the shear friction theory. The proposed design method produced a close and conservative estimate of lateral-load strength of two-way hinges.

2) Concrete in the hinge region was capable of sustaining large strains because it was well confined by both the hinge lateral steel and the column itself.

3) The dynamic testing of the specimens showed that, when cracks develop over the entire hinge section, significant shear slippage can take place between the column and footing thus reducing the energy absorption capacity of the member.

4) The dowel action of the hinge reinforcement prevented total failure of the hinge. Note that the dowel action is realized only after large slippage of the hinge, and hence it should not be used as the mechanism to determine the hinge shear capacity.
References

1. ACI Committee 318, “Building code requirements for structural concrete (318-99) and commentary (318R-99).” American Concrete Institute, Farmington Hills, Michigan, 1999.


Table 1: Summary of Two-Way Hinge Project Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Height (inch)</th>
<th>Column Diameter (inch)</th>
<th>Column Steel Ratio</th>
<th>Hinge Diameter (inch)</th>
<th>Area Ratio (A_h/A_c)</th>
<th>Hinge Steel Ratio</th>
<th>Axial Load (kips)</th>
<th>Axial Load Ratio (P/A_c*0.1f_c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>THD-1</td>
<td>48&quot;</td>
<td>16&quot;</td>
<td>4.20%</td>
<td>10</td>
<td>0.39</td>
<td>1.0%</td>
<td>107</td>
<td>10.0%</td>
</tr>
<tr>
<td>THD-2</td>
<td>64&quot;</td>
<td>16&quot;</td>
<td>3.88%</td>
<td>10</td>
<td>0.39</td>
<td>1.0%</td>
<td>60</td>
<td>6.0%</td>
</tr>
<tr>
<td>THD-3</td>
<td>48&quot;</td>
<td>16&quot;</td>
<td>1.40%</td>
<td>10</td>
<td>0.39</td>
<td>1.0%</td>
<td>107</td>
<td>10.0%</td>
</tr>
<tr>
<td>THD-4</td>
<td>48&quot;</td>
<td>16&quot;</td>
<td>1.50%</td>
<td>10</td>
<td>0.39</td>
<td>1.0%</td>
<td>7</td>
<td>0.70%</td>
</tr>
<tr>
<td>THD-5</td>
<td>48&quot;</td>
<td>16&quot;</td>
<td>3.0%</td>
<td>8</td>
<td>0.25</td>
<td>1.31%</td>
<td>107</td>
<td>10.0%</td>
</tr>
</tbody>
</table>

Table 2: Evaluation of Friction Coefficient ($\mu$) Using Test Data

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Lateral Force (V)</th>
<th>Axial Load (P)</th>
<th>Force at Compression Steel (Cs)</th>
<th>Force at Tension Steel (T)</th>
<th>Total Compression Force (Cs+C_c)</th>
<th>$\mu = \frac{F}{(Cs+ C_c)}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>THD-1</td>
<td>84.3</td>
<td>109.4</td>
<td>16.7</td>
<td>47.2</td>
<td>156.6</td>
<td>0.54</td>
</tr>
<tr>
<td>THD-2</td>
<td>53.0</td>
<td>70.3</td>
<td>18.9</td>
<td>46.4</td>
<td>116.9</td>
<td>0.45</td>
</tr>
<tr>
<td>THD-4</td>
<td>38.1</td>
<td>6.9</td>
<td>9.1</td>
<td>65.7</td>
<td>72.6</td>
<td>0.52</td>
</tr>
<tr>
<td>THD-5</td>
<td>67.7</td>
<td>114.1</td>
<td>28.6</td>
<td>34.6</td>
<td>148.7</td>
<td>0.46</td>
</tr>
</tbody>
</table>

Average $\mu$ : (THD1,THD2,THD4 and THD5) 0.49

Standard deviation: (THD1,THD2,THD4 and THD5) 4%

Note:
1) THD-3 specimen is excluded in the table, due to its column flexural failure mode.
2) For the design purpose, a $\mu$ factor of 0.45 is recommended.

Table 3: Two-Way Hinge Shear Capacity Prediction Using Different Methods

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Measured Shear Capacity</th>
<th>Shear Friciton Method</th>
<th>Diagonal Shear Method (beam shear)</th>
<th>UNR Method ($\mu = 0.45$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>THD-1</td>
<td>84.3</td>
<td>166.4</td>
<td>230.7</td>
<td>172.2</td>
</tr>
<tr>
<td>THD-2</td>
<td>53.0</td>
<td>132.7</td>
<td>163.1</td>
<td>133.1</td>
</tr>
<tr>
<td>THD-4</td>
<td>38.1</td>
<td>70.0</td>
<td>95.5</td>
<td>69.7</td>
</tr>
<tr>
<td>THD-5</td>
<td>67.7</td>
<td>168.2</td>
<td>167.5</td>
<td>154.3</td>
</tr>
</tbody>
</table>

Note:
All the values in table are in Kips.
Figure 1: Typical Specimen Detail (Specimen THD-1)

Figure 2: Typical Shake Table Setup for Two-way Hinge Project
Figure 3: Typical Input Earthquake Motion for Two-Way Hinge Project

Figure 4: THD-1 Force Displacement Relationship

Figure 5: THD-2 Force Displacement Relationship
Figure 6: THD-3 Force Displacement Relationship

Figure 7: THD-4 Force Displacement Relationship

Figure 8: THD-5 Force Displacement Relationship
Figure 9: Idealized Force Displacement Curves

Figure 10: Lateral Force vs. Hinge Slippage Envelope for All Specimens
Figure 11: Two-Way Hinge Failure Mechanism under Combine Loading

Figure 12: Two-Way Hinge Structural Drawing for Example Problem