

Behavior of Reinforced Concrete-Filled Steel Tube Columns with External Socket Connections Under Seismic Loading

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Submitted: 14 August 2026 Accepted: 17 September 2026 Publication date: 10 October 2026

DOI: 10.70465/ber.v3i4.106

Abstract: This study evaluates a new external socket connection for reinforced concrete-filled steel tube (RCFST) columns intended for seismic applications. The connection incorporates a steel-tube discontinuity at the column-to-socket interface, where the main tube is intentionally terminated, and a thinner embedded sleeve is used to reduce moment transfer into adjoining elements and facilitate plastic-hinge formation away from the connection. Two full-scale specimens were tested under reversed cyclic loading to assess global behavior, failure modes, strain demands, displacement capacity, and energy dissipation. The specimens had identical column geometry, while socket height, column concrete strength, and grout strength were selected using an analytical framework to produce two connection configurations with distinct predicted failure modes. One specimen exhibited flexural failure with hinge formation in the column outside the external socket, whereas the other developed shear failure in the portion of the column embedded within the socket, consistent with the respective analytical predictions. Strain measurements indicated limited localized yielding of the external socket wall reinforcement and steel sleeve in the shear-governed specimen, while these components remained elastic in the flexural specimen. No visible damage was observed in the external socket wall or footing in either test, supporting the intended capacity-protected behavior of these components. The results provide experimental support for the analytically predicted failure mechanisms for the two configurations investigated and demonstrate that the tested connections developed the calculated nominal moment capacity of the corresponding cast-in-place columns.

Author keywords: Reinforced concrete-filled steel tube; external socket connection; large-scale tests; seismic design; bridge design

Introduction

Reinforced concrete-filled steel tube (RCFST) columns are widely used in bridge construction, particularly within regions of high seismicity.^{1–4} These systems consist of a steel tube filled with concrete and internal longitudinal and transverse reinforcement, forming a composite section that provides enhanced strength and ductility under lateral loading when compared with conventional reinforced concrete or steel piers.^{5–8} RCFST columns offer several advantages, including their ability to function simultaneously as a deep foundation element below ground and as a column above

ground, the use of the steel tube as permanent formwork that expedites construction, and the high level of confinement provided by the tube, which increases flexural strength and deformation capacity while exhibiting minimal levels of damage.^{1,3,4,6–10} An example of their implementation is shown in Fig. 1 for the O'Malley Bridge in Alaska,¹¹ where the RCFST serves continuously as both the foundation and the above-grade columns.



Figure 1. Example of an RCFST multicolumn bent used in the O'Malley Bridge, AK

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Discussion period open till six months from the publication date. Please submit separate discussion for each individual paper. This paper is a part of the Vol. 3 of the International Journal of Bridge Engineering, Management and Research (© BER), ISSN 3065-0569.

For RCFST pile-column systems, the geometric configuration can vary, with some applications using an oversized below-grade pile relative to the above-grade column, while others adopt a constant diameter for the pile and column.^{6,12,13} Regardless of the geometric configuration, these systems can develop plastic hinges under lateral loading, either below grade or above grade. Under transverse loading in a multi-column bent, a plastic hinge commonly forms at the column–cap beam interface due to the fixed-head boundary, whereas longitudinal loading may induce hinging below grade.^{1,3,4,14,15}

Focusing on the interface between the column and the adjoining structural element, RCFST columns are often terminated just below this element, creating a gap or discontinuity in the steel tube between the column and the adjacent member.^{1,3,4,16} This discontinuity is intended to limit moment transfer, such that the connection to the superstructure is achieved through a reinforced concrete region, thereby facilitating the design of the cap beam or foundation as a capacity-protected element.

For seismic design, it is essential to detail the connection between the RCFST column and the adjoining structural elements so that plastic demands are shifted away from the cap beam or footing, ensuring these components remain capacity protected and that the plastic hinge develops at the intended discontinuity in the RCFST. However, strain penetration will still occur at the interface, increasing the likelihood of cracking in the adjoining elements. These considerations have motivated the study and development of an alternative connection system capable of relocating plastic demands away from protected components while maintaining reliable seismic performance. External socket connections provide a feasible solution by ensuring that inelastic demands concentrate outside the connection region and preserving the capacity-protected behavior of the adjoining structural elements.¹⁷

An external socket connection consists of an external wall integrated with the cap beam or footing, forming a voided opening into which the column is inserted. The column and the external socket wall are bonded using cast-in-place concrete or grout, eliminating the need for additional reinforcement within the connection.¹⁷ These connections simplify the construction of adjoining members compared to internal socket connections by reducing steel congestion in the cap beam or foundation and allowing for smaller cap beam cross-sections, since the void is located outside the structural element. These characteristics make external socket connections convenient for use in cap beams while also complementing their established application in footings for internal socket connections.^{18–22} Despite the advantages of external socket connections, no research to date has examined their seismic behavior when used with RCFST columns, whereas only limited studies have investigated the seismic performance of RCFST columns with internal socket connections.¹⁶

Understanding the system behavior is essential for developing connection details that ensure the plastic hinge will develop outside the connection while preserving the structural integrity of the adjoining elements, achieving seismic

performance comparable to that of a cast-in-place RCFST column. Although the development of an analytical model is beyond the scope of this paper, analytical evaluations informed the selection of key design parameters of the external socket connection, such as socket height (H_s), external socket wall thickness (t_w), and grout strength ($f'g$), for the experimental program.²³

This paper introduces a new connection detail for RCFST columns that use external socket connections for seismic applications. The detail mimics the RCFST-to-cap beam connection employed by AKDOT&PF^{4,14} and is adapted for a socket connection. This study evaluates the seismic performance of RCFST columns with external socket connections through two large-scale experimental tests with identical RCFST column geometry but different socket heights, column concrete strengths, and grout strengths. The two configurations were designed to develop distinct failure modes and assess their overall seismic response.

The specimen parameters were selected based on analytical evaluations,²³ which were developed using results from six previous experimental tests of RC columns with socket connections.¹⁷ The previous study identified socket height as a primary design variable, with the required socket height also dependent on the material properties of the column and grout. In the present study, the analytical framework was used to select two RCFST socket connection configurations intended to develop distinct failure modes. Therefore, the specimens were not designed to independently evaluate the effects of socket height, column concrete strength, or grout strength, but rather to experimentally assess the response of RCFST columns for two connection configurations associated with different predicted failure modes. The system performance is quantified in terms of lateral deformation capacity, hysteretic energy dissipation, and observed failure modes. The results provide experimental evidence of the response and failure mechanisms of the two selected connection configurations, including the conditions associated with plastic-hinge formation in the RCFST column outside the external socket connection, and provide insight into the implementation of RCFST columns with external socket connections in seismic regions.

Experimental Test Methodology

Two large-scale tests on RCFST columns with external socket connections were conducted at the Constructed Facilities Laboratory at North Carolina State University. The columns were connected to a footing or inverted cap beam using external socket connections and incorporated a steel discontinuity detail intended to reduce moment transfer into the connection. To implement this discontinuity, the steel tube of the RCFST column was terminated 51 mm before the interface with the external socket, and a separate steel tube with reduced thickness was installed for the portion embedded within the socket, as shown in Fig. 2. This thinner steel tube functioned primarily as formwork and provided confinement but it did not contribute significantly to flexural

strength due to the limited bond between the concrete and the steel tube.⁸

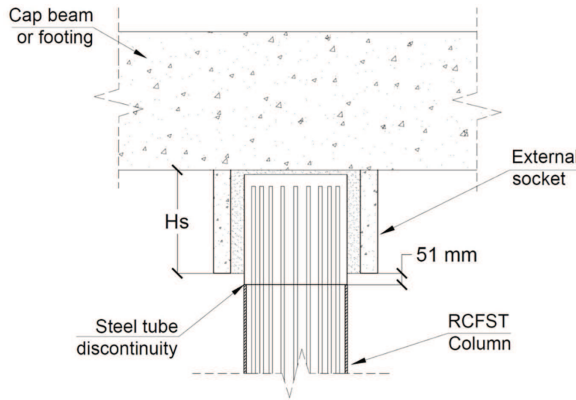


Figure 2. Discontinuity detailing of the RCFST column

For both large-scale specimens, the thickness of the thinner tube was one-third that of the main RCFST column tube, ensuring that the embedded sleeve was weaker than the primary steel tube. This connection detail ensured that the moment transferred to the external socket and footing or cap beam was governed by the confined reinforced concrete section alone, rather than by the full composite capacity of the RCFST column.

The differences between the two experiments are summarized in Table 1 and focus on variations in socket height and the material strengths of the grout and column concrete, while the RCFST column detailing was kept constant for both specimens. The specimen configurations were selected using an analytical framework,²⁴ which relates the socket geometry and material strengths to the nominal flexural capacity of the column through three potential governing mechanisms. The analytical framework is summarized herein through Eqs. (1) to (3) to clarify the basis used for the specimen design. Eq. (1) represents the condition associated with flexural failure of the column, Eq. (2) represents the shear failure mechanism and incorporates the revised UCSD shear model,²⁵ and Eq. (3) represents the failure mechanism governed by the grout. For a given set of geometric and material properties, each equation provides the required socket height associated with its corresponding failure mechanism, and the largest required socket height governs the connection design.

$$\chi \geq \frac{0.6 * P * D_c + 1.2 * M_n}{0.1 * f'_c * D_c^3} \quad (1)$$

$$\chi \geq \frac{M_n}{1.3 * \left(\frac{2 * A_{sp} * f_{yd} * D_c^2}{S} + 0.21 * \sqrt{f'_{cd}} * D_c^3 \right)} [MPa] \quad (2)$$

$$\chi \geq 3.7 * \sqrt{\frac{M_n}{f'_{cg} * D_c^3}} \quad (3)$$

Where P = Axial force acting on the column; Mn = Nominal moment capacity; Dc = Diameter of the column; f'_{cd} = Design concrete compressive strength; f'_{cg} = Design grout compressive strength (Cylinder); f_{yd} = Design yield stress; A_{sp} = Area of spiral for transverse reinforcement;

S = Spacing of transversal reinforcement; and χ = Ratio between socket height and diameter of the column.

Using this framework, two connection configurations were selected to experimentally evaluate distinct expected failure modes: one specimen was designed to develop a flexural failure mode, with the plastic hinge forming in the RCFST column outside the socket connection, while the second specimen was designed to develop a shear-governed failure mode. The nominal flexural capacity of the RCFST column was maintained the same for both specimens, while the socket height and the grout and column concrete strengths were selected using the analytical framework to obtain the targeted response. The analytical design parameters, required socket heights, and expected failure modes for both specimens are summarized in Table 1. Therefore, the variations in socket height and material strengths represent components of each analytically designed connection configuration rather than independent experimental variables.

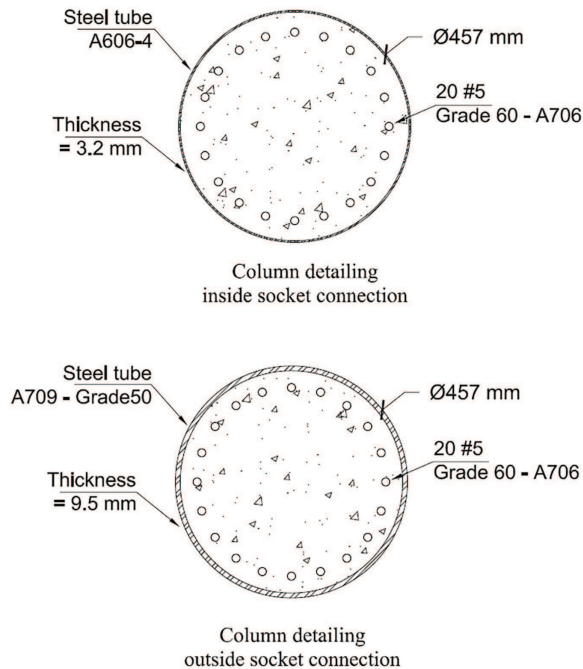
The RCFST column specimens, SJS7 and SJS8, had a clear height of 2286 mm and a diameter of 457 mm, resulting in a height-to-diameter ratio of 5.0. The longitudinal reinforcement consisted of 20 Grade 60-A706 #5 steel bars (16 mm nominal diameter), corresponding to a reinforcement ratio (ρ_l) of 2.4%. The external steel tube was made of A709 Grade 50, which provided confinement and shear capacity to the concrete and had a thickness of 9.5 mm. The concrete core did not contain transverse reinforcement. The clear distance between the outer surface of the longitudinal reinforcing steel and the inner surface of the steel tube was 25 mm.

The portion of the column embedded in the external socket maintained the same longitudinal and transverse reinforcement details as the section outside the socket. However, the steel tube within the socket region was made of A606-4 and had a thickness of 3.2 mm, equivalent to one-third of the thickness of the steel tube outside the socket. Within the socket region, this thin steel tube primarily served as permanent formwork and was not considered to provide additional shear capacity to the embedded portion of the column. Fig. 3 shows the plan view of the RCFST column inside and outside the external socket connection, while Fig. 4 presents the elevation view of the RCFST column detailing.

The longitudinal reinforcement of the external socket wall for both specimens consisted of 20 Grade 80-A706 #6 steel bars (19 mm nominal diameter), anchored to the footing with standard hooks and extended to the level of the bottom reinforcement of the footing. The external socket was encased by steel sleeves made of A36 steel with a thickness of 6.3 mm. The thickness of the external socket wall (tw) was 83 mm, and the gap between the column and the external socket was 25 mm. The concrete compressive strength of the external socket wall was 77.8 MPa for SJS7 and 96.5 MPa for SJS8. The footing and cap beam were designed as capacity-protected components and featured identical detailing for all specimens. Figs. 5 and 6 illustrate the reinforcement details of the external socket connection.

Table 1. Specimen parameters and predicted failure mechanisms

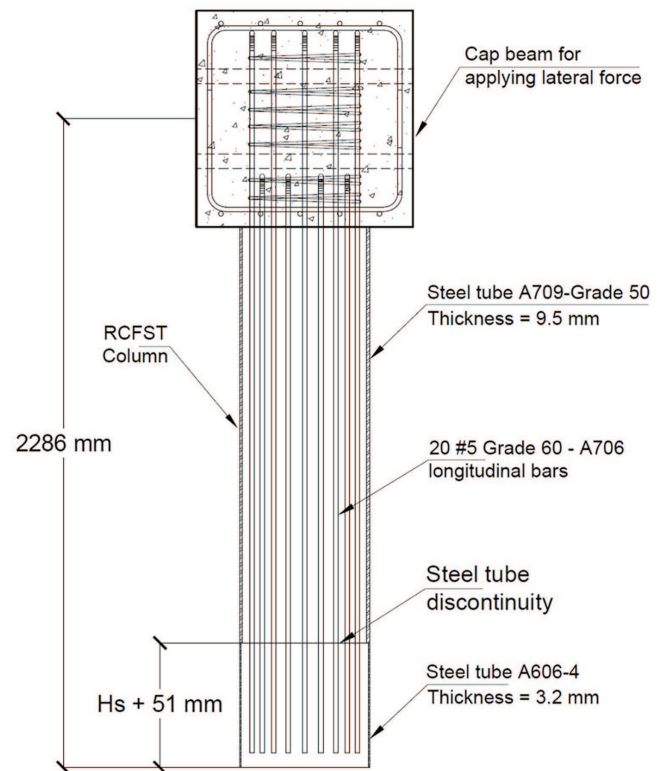
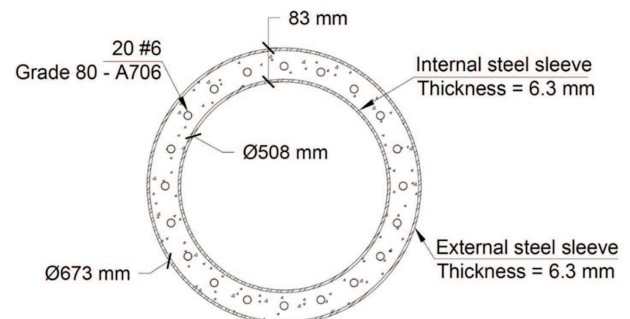
Test specimen	Column diameter [D_c] (mm)	Hs (mm)	Hs(% D_c)	Column f'_c (MPa)	Grout f'_g (MPa)	Predicted failure mechanism
SJS7	457	389	0.85 D_c	38.6	62.5	Shear
SJS8	457	559	1.22 D_c	48.3	40.1	Flexural

**Figure 3.** Plan view of RCFST column

Concrete and grout compression tests were conducted on 100 mm x 200 mm cylinders following ASTM C39.²⁶ This standard was selected because most grout mixes in the experimental program included a coarse aggregate, making ASTM C39 more appropriate for assessing their compressive strength. A constant axial load equal to 5% of the design axial column capacity (343 kN) was applied to all specimens. This percentage value was chosen to maintain consistency with previous tests conducted at NCSU and is considered a lower-bound axial load level representative of minimum gravity load demands.

Test setup and instrumentation

SJS7 and SJS8 were tested under lateral loading using a 978-kN hydraulic actuator with a 1016 mm stroke, secured to a strong wall through a steel plate. The specimens were positioned at the actuator's mid-stroke. An axial load ratio of 5% of the design axial column capacity was applied to both specimens. This axial load ratio was selected to maintain consistency with previous column tests conducted at NCSU, including the six RC columns with external socket connections used in developing the analytical framework, and to facilitate comparison with the existing experimental database. The 5% axial load ratio also represents a

**Figure 4.** Elevation view of RCFST column**Figure 5.** Plan view of external socket detailing

lower-bound gravity demand consistent with the bridge applications considered in the experimental program. A spreader beam supporting two 490-kN hydraulic jacks, placed above the cap beam, was used to apply the constant axial load during testing. A self-regulated axial load system was used to maintain the constant axial force, incorporating a third jack within the Universal Testing Machine operating under force-controlled mode.²⁷

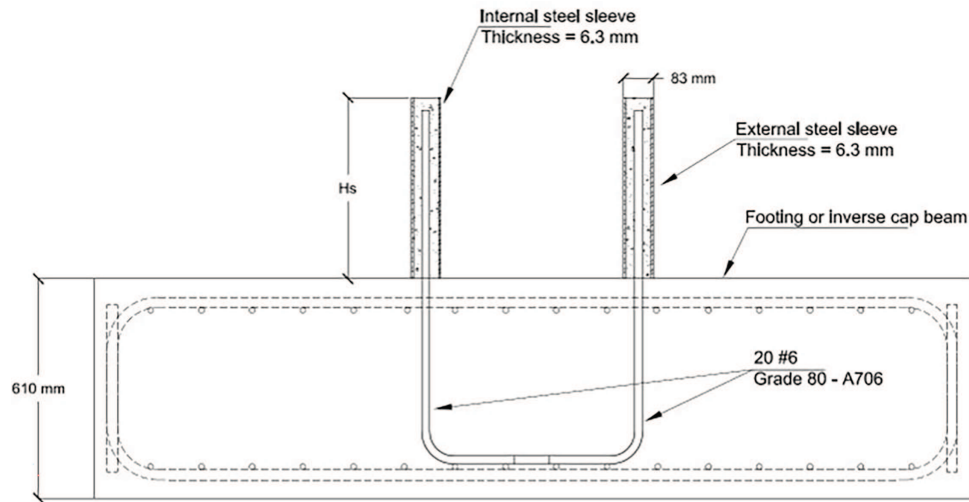


Figure 6. Elevation view of external socket detailing

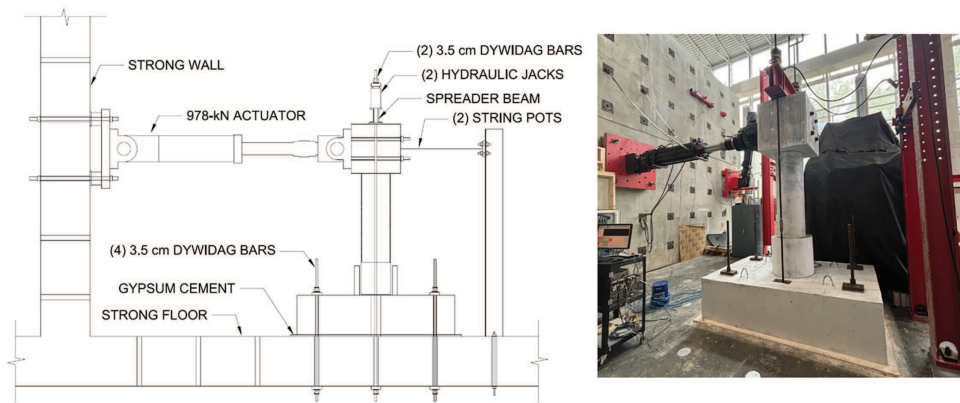


Figure 7. Experimental test setup of RCFST columns

Gypsum cement was applied between the footing and the strong floor to enhance surface contact and friction. Additionally, the foundation was anchored to the strong floor using four 3.5 cm diameter bars, each post-tensioned to 445 kN. Specimen displacement was monitored with two string potentiometers, positioned at the center and edge of the cap beam to detect any potential rotation. Fig. 7 presents the test setup.

Digital Image Correlation (DIC) was employed to monitor the displacement field and strains in the steel tube of the RCFST column and the external socket sleeve. The DIC data was also used to determine the rotation of the external socket and the RCFST column. Additionally, 12 strain gauges were installed on the longitudinal reinforcement within the external socket wall to measure axial strains and verify the transmission of strains to the footing or cap beam.

Loading protocol

The specimens were subjected to a reverse cyclic unidirectional quasi-static load applied at the top of the column. Positive force and displacement indicated a push toward the north direction of the specimen, while negative values represented a pull toward the south direction.

The loading protocol for both experimental tests consisted of a single cycle at 25%, 50%, 75%, and 100% of the theoretical first yield force of the longitudinal column reinforcement. Beyond the yield point, three cycles were applied at progressively increasing displacement ductility levels. Displacement ductility of one was determined by extrapolating the experimentally measured first yield displacement (Δ'_y) to the nominal moment capacity of the cross-section, as defined in Eq. (4), where M'_y represents the theoretically calculated first yield moment of the column section. It is important to note that the measured first yield displacement accounts for various deformation components, including shear and flexural deformation of the column, strain penetration of the longitudinal column reinforcement into the embedded portion within the socket, rocking of the column, as well as shear deformation and rotation of the external socket.

$$\mu_1 = \frac{M_N}{M'_y} \Delta'_y \quad (4)$$

The displacement for any given ductility level “n” was calculated by multiplying the displacement at ductility one by “n,” as shown in Eq. (5).

$$\mu_n = n * \mu_1 \quad (5)$$

The tests continued until the recorded laterally applied force dropped by approximately 40% from the maximum observed value. Extending the lateral loading beyond this point was considered unnecessary, as it would likely not provide additional valuable information and could potentially compromise the safety of the test setup. Fig. 8 presents an example of the loading protocol used in the experimental tests.

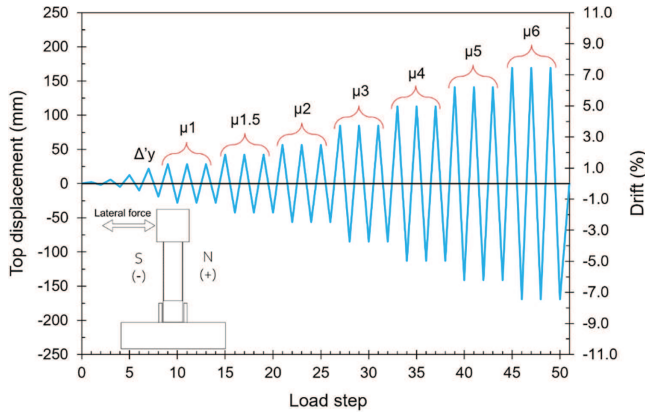


Figure 8. Example of loading protocol

Experimental Results and Discussion

Global response and associated failure modes

The global response of the two RCFST large-scale tests was evaluated using moment-displacement curves. These curves highlight key characteristics such as stiffness, ductility, and strength degradation. Fig. 9 presents the moment-displacement response for both specimens and includes a comparison with the nominal moment capacity of the corresponding CIP RC section at the steel tube discontinuity, calculated using the actual specimen geometry and measured material properties. While no cast-in-place (CIP) column was physically tested in this study, previous large-scale tests on CIP RCFST columns^{1–3,6,14} provide a basis for comparison. These CIP columns typically incorporate a gap at the interface with the footing, similar to the steel tube discontinuity applied in this study. At the discontinuity, the RCFST column transitions to behave as a reinforced concrete (RC) column, with strength governed by the fully confined RC core, independent of any contribution from the steel tube.⁸

While both SJS7 and SJS8 developed flexural plastic hinges in the column region above the external socket connection, their failure mechanisms differed. SJS7 exhibited a shear failure within the portion of the column embedded in the socket after reaching a peak moment of 428 kNm at a displacement ductility of five, which was approximately 4.6% higher than the nominal flexural capacity of a CIP column. This failure mode was identified based on consistency with the observed response of other external socket connection specimens that exhibited shear failure,¹⁷ as well as post-test inspection of the RCFST column within the socket,²³ where distinct shear cracking was observed.

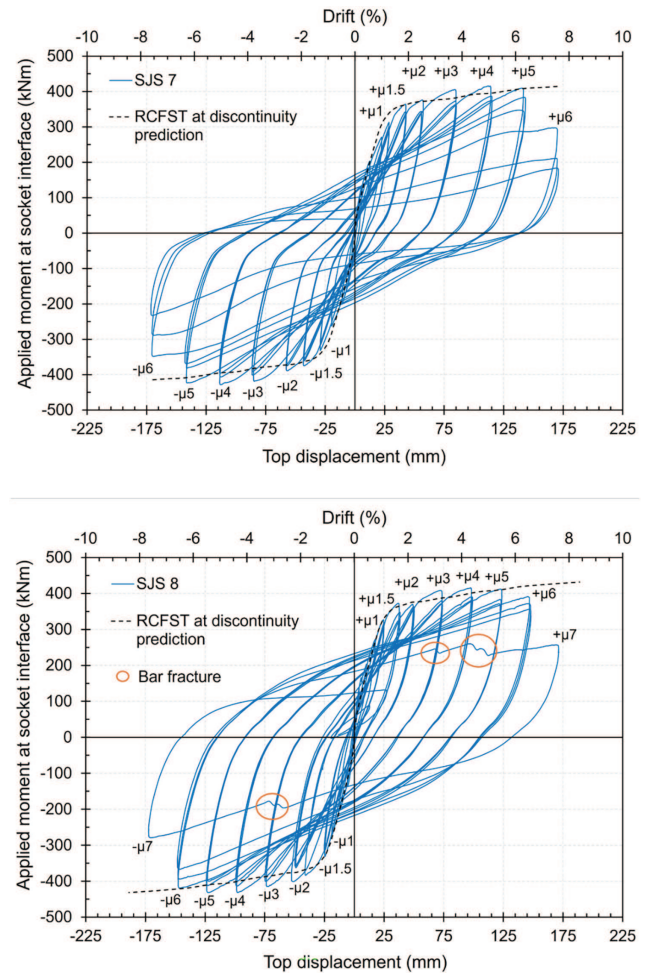


Figure 9. Measured moment–displacement response at external socket interface

As shown in Fig. 9, a drop of the maximum applied moment of approximately 55% occurred during the second cycle of ductility six, coinciding with extensive concrete core crushing and column rocking, leading to the conclusion of the test after the third cycle of ductility six. Although a plastic hinge developed on the column at the socket interface, longitudinal bar fracture did not occur, as the shear failure of the column inside the external socket connection prevented the development of sufficient force to cause bar rupture of the longitudinal steel of the column.

In contrast, SJS8 experienced a flexural failure characterized by longitudinal bar fracture above the socket connection. The specimen reached a peak moment of 432 kNm, which was approximately 3.3% higher than the CIP nominal capacity at a displacement ductility of seven. The observed response is consistent with the behavior reported for other socket connection specimens exhibiting a flexural failure mechanism.^{17,19,20,22} As shown in Fig. 9, a drop of the maximum applied moment of approximately 45% was observed during the first cycle of ductility seven. At this stage, three longitudinal bars fractured on the south side during the first positive cycle of ductility seven, followed by two additional bar fractures on the north side during the first negative cycle, leading to the conclusion of the test. Due

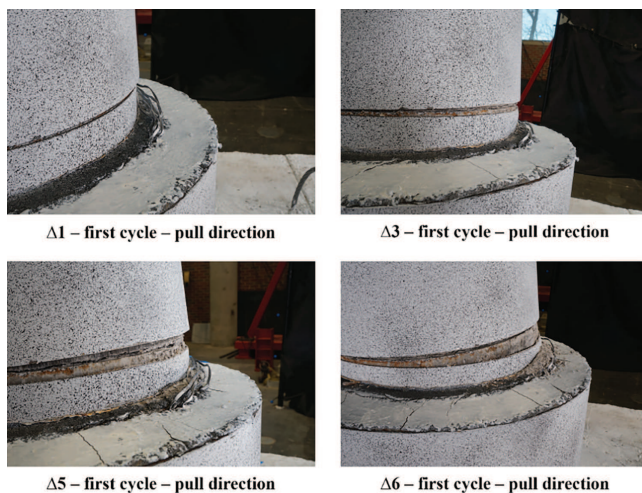
Table 2. Summary of global response of test SJS7 and SJS8

Test specimen	Column Mn at discontinuity (kNm)	Peak moment on column at socket interface (kNm)	Failure mode (reached limit state)	$\Delta \mu 1$ (mm)	Ductility at limit state	Drift at limit state
SJS7 (0.85 D_c)	409.1	428.0	Shear failure of the column inside the connection	28.2	$+\mu 5$	6.2%
SJS8 (1.22 D_c)	418.3	432.0	Flexural failure of the column. Bar fracture	24.4	$+\mu 7$	7.5%

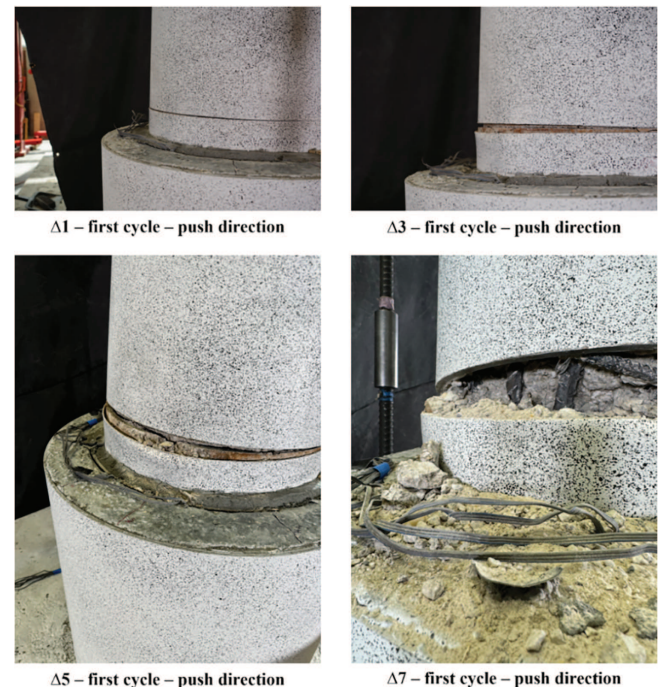
to the external steel sleeve, it was not possible to visually identify which bars fractured during testing.

Despite the differences in failure type, both specimens demonstrated the formation of plastic hinges outside the connection region, confirming the effectiveness of the external socket connection and steel discontinuity detail in enabling the desired inelastic mechanism. Table 2 summarizes the global response characteristics and complements the information presented in the moment-displacement curves.

The damage progression of SJS7 is illustrated in Fig. 10, which presents damage observations at displacement ductility of 1, 3, 5, and 6, focused on the north side of the column at the discontinuity interface during pull cycles. The images highlight the progressive increase in the gap between the steel tubes and the concrete damage of the column. At ductility one, a 4 mm gap was observed between the steel tubes. By the final cycle of ductility two, this gap increased to 9 mm, accompanied by local buckling of the thinner steel tube at the discontinuity interface as it underwent compression.

**Figure 10.** Damage progression for SJS7

As loading progressed, the gap between the steel tubes further increased, reaching 15 mm at ductility three and 22 mm at ductility four. Additionally, during the first cycle of ductility three, a 2 mm crack appeared in the concrete at the interface between the steel tubes. Strength degradation initiated at ductility five, coinciding with core crushing at the

**Figure 11.** Damage progression for SJS8

socket interface. By the final cycle of ductility five, the gap had increased to 30 mm, and visible concrete core crushing was observed at the interface between the steel tubes. During the last cycle of ductility six, the gap expanded to 42 mm, and the concrete core experienced extensive crushing at the interface between the steel tubes, marking the final stage of damage progression before the test was ended.

The damage progression of SJS8 is illustrated in Fig. 11, which presents damage observations at displacement ductility of 1, 3, 5, and 7, focused on the south side of the column during push cycles. The images highlight the progressive increase in the gap between the steel tubes and the concrete damage of the column. At ductility one, a 3 mm gap was observed at the discontinuity interface of the steel tubes. By the final cycle of ductility two, this gap increased to 11 mm, accompanied by local buckling of the thinner steel tube at the discontinuity interface as it underwent compression.

As the test progressed, the gap between the steel tubes continued increasing, reaching 15 mm at ductility three and 22 mm at ductility four. Additionally, during the first cycle

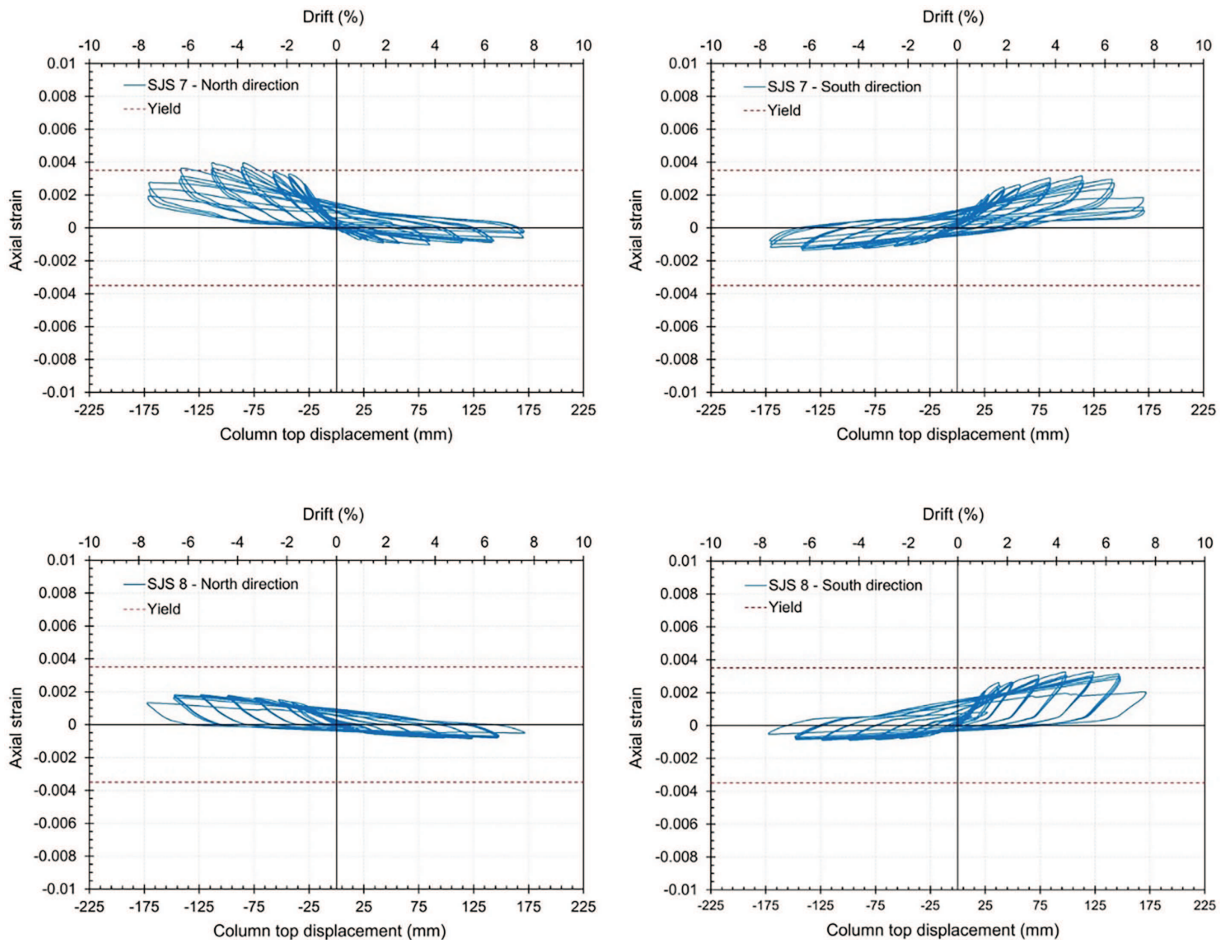


Figure 12. Axial strain of external socket reinforcement

of ductility three, a 2 mm crack appeared in the concrete at the interface between the steel tubes. By the final cycle of ductility five, the gap had increased to 30 mm, and visible concrete core crushing was observed at the interface between the steel tubes. During the last cycle of ductility six, the gap expanded to 42 mm, and the concrete core experienced extensive crushing at the interface between the steel tubes. At ductility seven, the column experienced three bar fractures on the south side and two additional fractures on the north side. Due to the external steel sleeve, it was not possible to directly observe which bars fractured during testing.

The formation of a gap at the discontinuity interface was observed in both tests and is an expected and desirable outcome. This ensures that the steel tube does not contribute to the flexural capacity of the RCFST column at the discontinuity interface and inside the external socket, allowing the forces transferred to the external socket connection to be considered only from the RC column with a fully confined cross-section.

For both specimens, the external socket wall did not sustain significant damage, with only minor radial cracks in the concrete and grout appearing before the first yield displacement of the RCFST column. The inner and outer steel sleeves of the socket did not exhibit any visible inelastic

deformations, and the footing remained undamaged, confirming its role as a capacity-protected element.

Axial strains and associated failure modes

For both experimental tests, the axial strain history of the extreme longitudinal reinforcement bars in the external socket wall was monitored using strain gauges placed 50 mm above the footing. Additionally, axial strains on the external socket sleeve and the steel tube of the RCFST column were determined from the DIC data. The measured strains in the steel sleeve and steel tube reflect different deformation modes: horizontal strains capture the circumferential deformations caused by hoop forces around the external socket, while vertical strains result from the flexural deformation of the socket wall. Tension tests were performed to quantify the stress–strain properties of the reinforcing steel, in accordance with ASTM A370.²⁸ The yield strain of the Grade 80 steel was determined as 0.0035 at the transition between the linear and nonlinear portions of the stress–strain curve, and the yield strain for the steel sleeves was determined as 0.0024.

Fig. 12 presents the strains in the external socket reinforcement along both the north and south sides of the column for both large-scale tests. The strain measurements were consistent with this observation. In SJS7, the external socket wall reinforcement at the interface with the footing

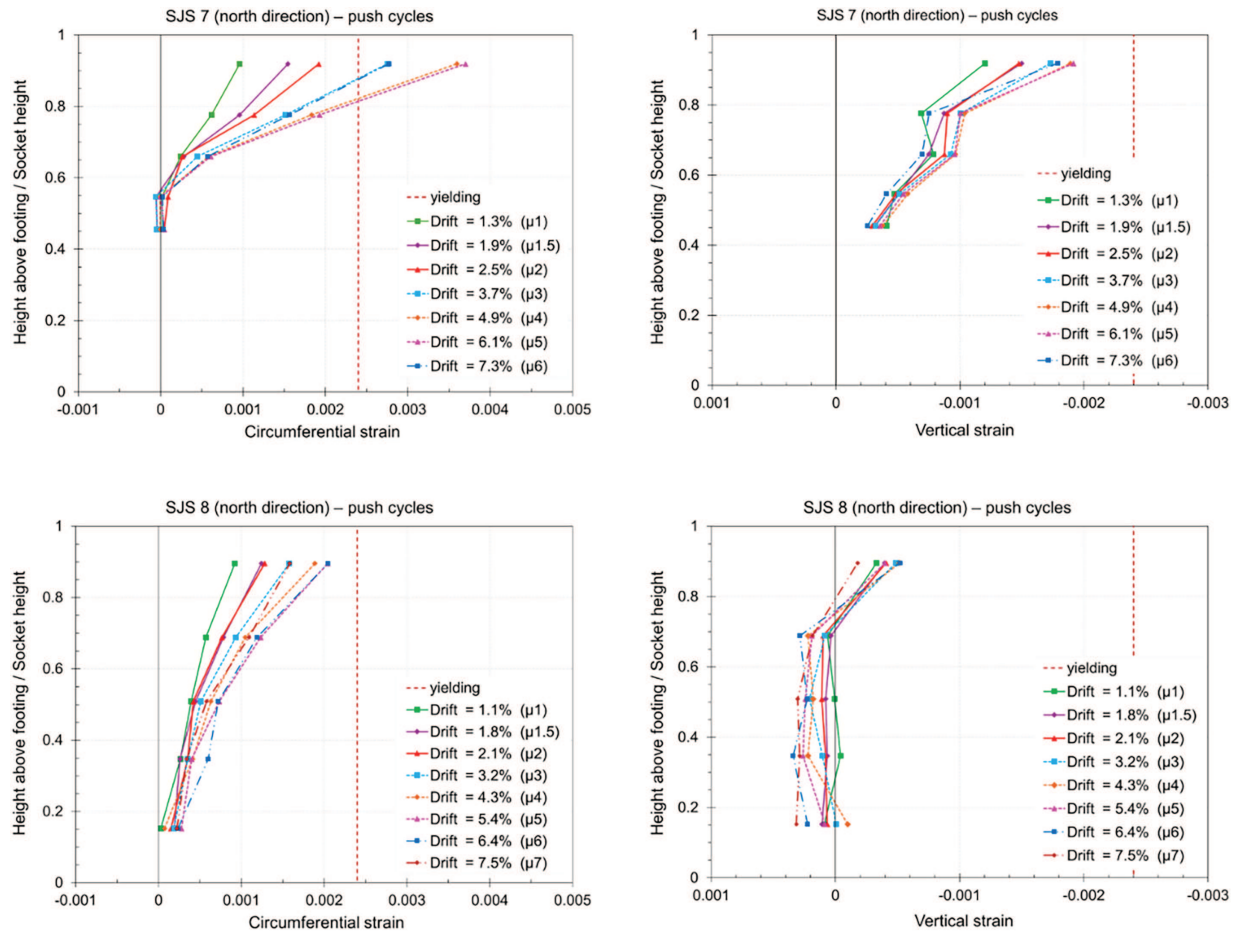


Figure 13. Circumferential and vertical strains on the external socket sleeve

reached approximately 1.15 times the yield strain, indicating localized yielding with limited plastic strain, while the measured strains in SJS8 remained below the yield strain. Therefore, although localized yielding occurred in the external socket wall reinforcement of SJS7, the limited exceedance of the yield strain was consistent with the absence of visible damage to the external socket wall. The external socket connections effectively confined plastic strains to the column, preventing them from extending into the foundation or cap beam, confirming their role as capacity-protected elements.

Following the analysis of reinforcement strains in the socket wall, strain measurements on the steel sleeves of both the external socket and the RCFST columns were evaluated. Fig. 13 presents the circumferential and vertical strains, measured along the height of the external socket sleeve. No visible damage was observed on the sleeves during either test, suggesting that they did not experience significant inelastic deformation.

In specimen SJS7, the circumferential strains reached approximately 1.5 times the yield strain within the top 20% of the socket height, indicating localized yielding of the steel sleeve in this region. Below this region, the measured circumferential strains remained below the yield strain. Vertical strains remained within elastic limits throughout the test,

indicating that the steel sleeve did not significantly contribute to the flexural resistance of the external socket wall.

For specimen SJS8, the highest circumferential strains also occurred in the top 20% of the socket height but remained below the yield strain, indicating fully elastic behavior across the entire sleeve. Vertical strains remained elastic during all loading cycles. Although compressive vertical strains were expected during push cycles, the recorded values were small and within the margin of measurement error, suggesting negligible flexural demand on the steel sleeve.

Fig. 14 presents the circumferential and vertical strains measured in the steel tube of the RCFST columns, just above the discontinuity interface. The data confirm that both circumferential and vertical strains remained elastic throughout the experiment. This behavior confirms that the steel tube above the discontinuity contributed primarily to confinement and shear capacity but did not significantly contribute to the flexural resistance. Consequently, the moment transferred to the external socket connection can be considered equivalent to that of a fully confined reinforced concrete column, because the discontinuity detail effectively isolates the steel tube from participating in the flexural resistance.

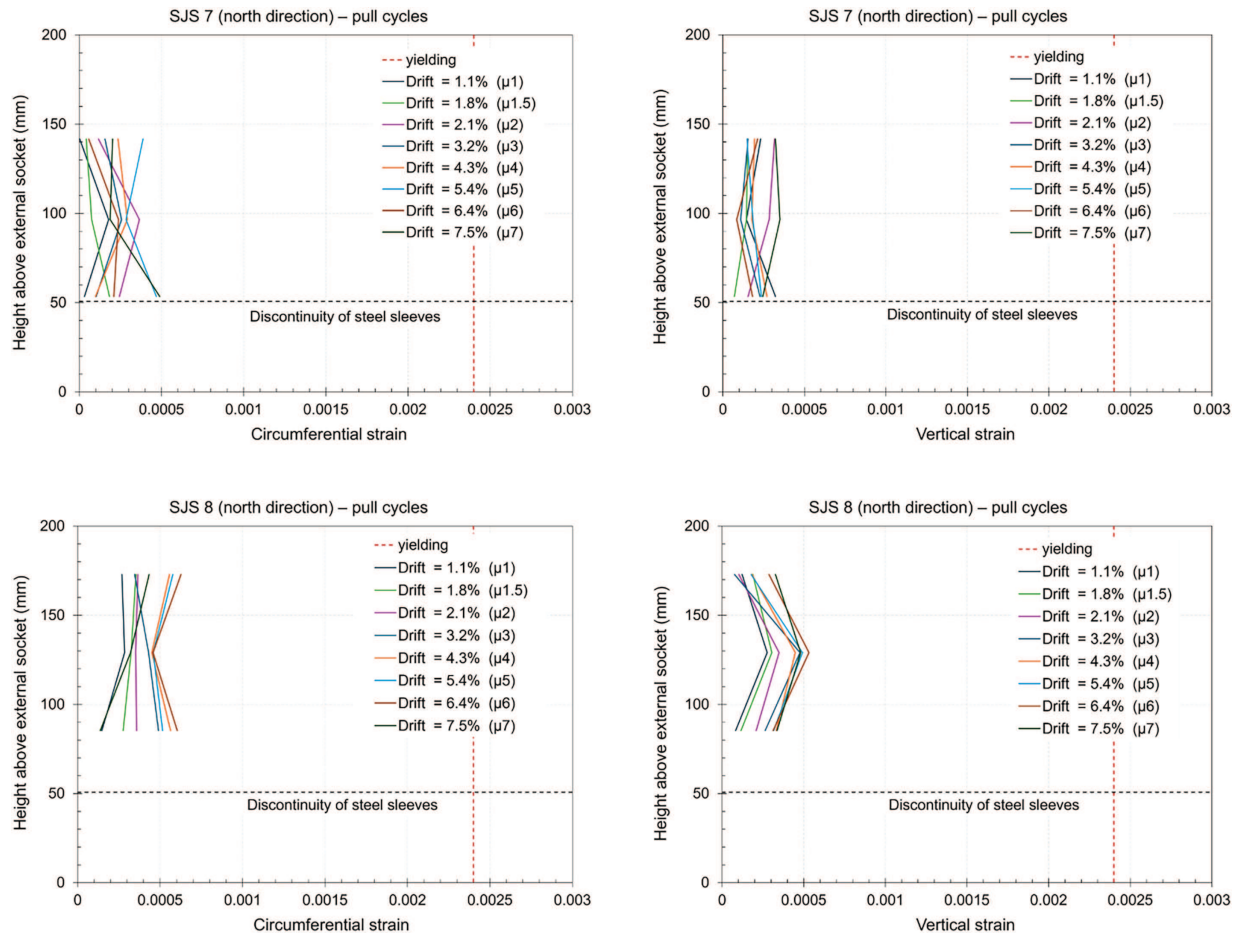


Figure 14. Circumferential and vertical strains on the Steel tube of the column

Hysteretic damping and associated failure modes

Hysteretic damping is a key parameter for evaluating how structures dissipate energy under large inelastic deformations, which is particularly relevant for assessing seismic performance. In this study, hysteretic damping was calculated using Jacobsen's area-based method,²⁹ which defines damping as the ratio of the area enclosed by the force–displacement hysteresis loop to the total elastic energy stored during the same loading cycle. The values reported herein correspond directly to the damping calculated using the Jacobsen method and do not represent inherent structural damping or total equivalent viscous damping.

Fig. 15 illustrates the hysteretic damping values for all cycles across the various ductility levels for specimens SJS7 and SJS8. As expected, damping increased with higher ductility levels, reflecting greater energy dissipation as inelastic deformation progressed. Higher hysteretic damping was observed during the cycles prior to first yield in both specimens compared with the 5% value commonly reported for conventional CIP RC columns and RCFST columns at the location of the discontinuity.^{27,30,31} This increase is attributed to initial cracking of the grout and socket wall, which allowed limited rocking at the connection before the

column reached first yield. The resulting rocking affected the shape and enclosed area of the measured hysteresis loops and is therefore inherently reflected in the damping calculated using the Jacobsen area-based method.

For specimen SJS7, the elastic damping ratio was approximately 14%, representing an increase of about 9% relative to typical CIP columns. For specimen SJS8, the elastic damping ratio was approximately 10%, also exceeding the CIP baseline, indicating enhanced energy dissipation in the elastic range for both cases.

Both specimens exhibited comparable hysteretic damping across ductility levels up to ductility six. At that stage, SJS7 began to experience shear failure and reached a peak hysteretic damping of approximately 28%, whereas SJS8, which exhibited flexural failure, reached a slightly higher peak damping of approximately 31%. The difference in damping behavior at higher ductility levels can be attributed to their distinct failure mechanisms. For SJS7, the increased damping was likely due to enhanced rocking and localized damage following shear failure. In contrast, for SJS8, the energy dissipation was primarily due to progressive flexural damage in the concrete and the eventual fracture of the column longitudinal reinforcement.

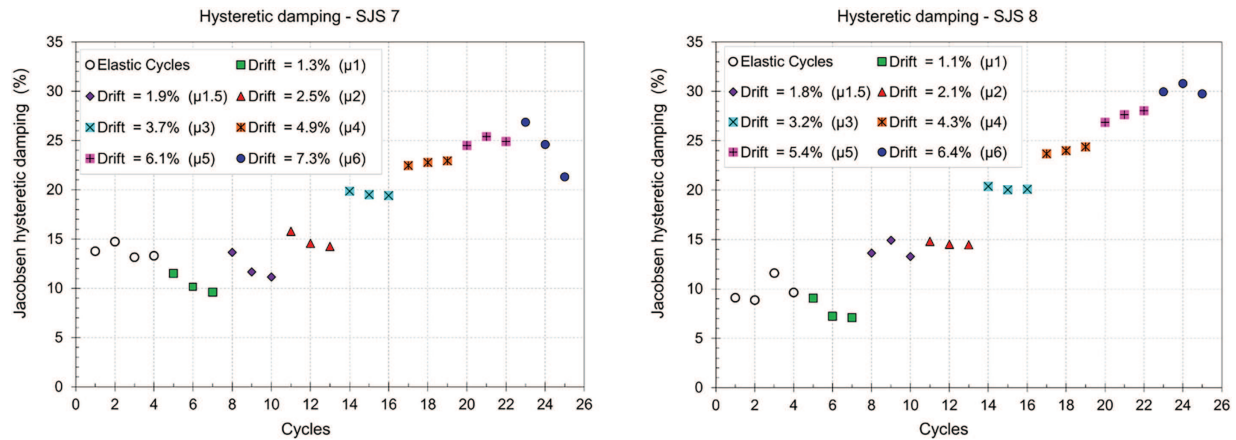


Figure 15. Hysteretic damping for all cycles of each ductility displacement

Summary of results and discussion

The experimental results from the two large-scale tests provide insight into the seismic behavior of RCFST columns using external socket connections. Both specimens developed a plastic hinge in the column outside the external socket connection; however, their failure modes and overall performance differed.

SJS8 demonstrated the preferred seismic behavior, with a flexural failure mechanism characterized by the fracture of the column longitudinal reinforcement above the external socket. This confirms the successful development of the plastic hinge outside the connection and represents a preferred and repairable failure mode. In contrast, SJS7 experienced a shear failure within the portion of the column embedded in the socket. Although a hinge formed above the connection, shear failure led to excessive rocking and strength degradation before bar fracture could occur, consistent with the observed damage and shear failure mechanisms reported in other external socket connection tests.¹⁷ This failure mode is difficult to identify during loading because the damage is confined to the embedded portion of the column within the socket and is most clearly confirmed through post-test inspection, thereby compromising post-earthquake inspection and reparability.

The experimental program was intentionally designed to produce distinct failure modes to evaluate the seismic behavior under different parameter combinations. The predicted failure mechanisms were established based on prior research on external socket connections¹⁷ and an analytical framework,²³ and the corresponding design variables for each specimen are summarized in Table 1. The analytical framework, summarized by Eqs. (1) to (3), was used to select combinations of socket height and material strengths associated with the targeted failure mechanisms. SJS8 was designed with a combination of socket height, column concrete strength, and grout strength intended to prevent the shear failure mechanism from governing and allow flexural failure to develop outside the socket connection. In contrast, the parameter combination selected for SJS7 was intended to produce a shear-governed response within the embedded

portion of the column. The experimentally observed failure modes were consistent with these analytical predictions.

In both specimens, a physical gap formed at the steel discontinuity interface and widened with increasing ductility, consistent with the intended interruption of the steel-tube contribution to the RCFST flexural capacity at the column-to-socket interface. Within the socket, the embedded steel tube primarily served as permanent formwork and was not considered to provide additional shear capacity, consistent with the analytical design assumption that the main steel tube did not provide continuous flexural resistance into the socket connection. Strain measurements showed localized yielding of the external socket wall reinforcement and steel sleeve in SJS7, with measured strains slightly exceeding the corresponding yield strains and indicating limited localized plastic demands, while these components remained elastic in SJS8. Despite the localized yielding in SJS7, no visible damage was observed in the external socket wall or footing, supporting the intended capacity-protected behavior of the socket connection.

Both specimens also demonstrated effective energy dissipation, with hysteretic damping during cycles prior to first yield reaching approximately 14% for SJS7 and 10% for SJS8, compared with the approximately 5% value typically reported for CIP columns. The higher values are attributed to early cracking of the grout and socket wall and the resulting limited rocking at the connection. As inelastic deformations progressed, the hysteretic damping calculated using the Jacobsen method increased further, with SJS8 reaching a peak of approximately 31% and SJS7 reaching 28%, indicating considerable energy dissipation in both specimens despite their different failure modes. The measured moment capacity of both specimens exceeded the calculated nominal moment capacity of the corresponding CIP column, with SJS7 and SJS8 reaching approximately 5% and 3.3% above the calculated values, respectively. These results indicate that, when properly detailed with appropriate socket height and material strengths, the external socket connection can develop the nominal flexural capacity of the corresponding CIP column.

These findings provide experimental support for the use of the analytical framework to select combinations of socket height and material strengths associated with different failure mechanisms. For the two configurations investigated, the selected combinations of socket height, column concrete strength, and grout strength resulted in observed failure modes consistent with the analytical predictions. The results therefore emphasize the importance of considering these parameters together when designing the external socket connection to promote plastic hinge formation outside the connection and prevent a shear-governed response within the embedded portion of the column.

Conclusions

The two large-scale tests demonstrated that the tested RCFST columns with external socket connections developed the calculated nominal moment capacity of the corresponding CIP column, although the specimens developed different failure mechanisms. A key outcome of the study was the successful implementation of a new discontinuity detail at the column-to-socket interface intended to reduce moment transfer into the connection. This was achieved by terminating the RCFST steel tube before the interface and introducing a separate, thinner steel tube segment inside the socket. The embedded steel tube primarily served as permanent formwork and was not considered to provide additional shear capacity to the embedded portion of the column. The thinner embedded tube was selected to remain weaker than the main RCFST tube such that, upon contact between the two steel tube segments at the discontinuity, premature buckling of the main tube would be avoided. A gap formed at the discontinuity during testing, consistent with the intended interruption of the RCFST flexural capacity at the column-to-socket interface, such that the flexural contribution of the main steel tube was not transferred into the socket connection. The external socket wall reinforcement and steel sleeve exhibited limited localized yielding in SJS7 and remained elastic in SJS8. Despite the localized yielding, no visible damage was observed in the external socket wall or footing. Both specimens also exhibited considerable hysteretic energy dissipation. The hysteretic damping calculated using the Jacobsen method reached approximately 14% for SJS7 and 10% for SJS8 during cycles prior to first yield and increased with inelastic deformation to peak values of approximately 28% and 31%, respectively.

The observed failure modes were consistent with the predictions of the analytical framework used to select the socket height and material strengths. SJS8 developed the intended flexural failure mode above the socket, including fracture of the column longitudinal reinforcement and formation of a plastic hinge, whereas SJS7 developed the predicted shear failure in the embedded portion of the column. These results provide experimental support for the analytical framework for the two configurations investigated and demonstrate the importance of considering socket height and material strengths together when establishing the governing connection failure mechanism.

The experimental findings are limited to the two connection configurations investigated and an axial load ratio of 5% of the design axial column capacity. Although the analytical framework accounts for axial load in the connection design, the response of the proposed connection at higher axial load ratios was not experimentally evaluated. In addition, both specimens used an embedded sleeve thickness equal to one-third of the main RCFST steel tube thickness; therefore, this ratio should not be interpreted as a minimum or optimized design value. Further experimental investigation is required to evaluate the connection under higher axial demands and to establish the influence and appropriate thickness of the embedded sleeve.

Overall, the two tests provide experimental evidence that the proposed external socket connection and steel-tube discontinuity detail can develop the intended flexural or shear response predicted by the analytical framework while protecting the external socket wall and footing from visible damage. The results provide experimental support for the proposed connection concept within the range of parameters investigated and establish a basis for further validation of RCFST external socket connections for seismic bridge applications.

Acknowledgments

This project was funded by the Alaska Department of Transportation and Public Facilities. Their support is gratefully acknowledged. The authors sincerely appreciate the support and contributions from the faculty, students, and technical staff at the Constructed Facilities Laboratory at North Carolina State University.

Data Availability Statement

The data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request.

Supplemental Materials

The authors have declared that there are no supplemental materials associated with this manuscript.

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