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EXPERIMENTAL PERFORMANCE OF MOMENT CONNECTIONS IN CFT COLUMN-WF BEAM STRUCTURAL SYSTEMS UNDER SEISMIC LOADING

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ABSTRACT

The use of concrete filled tube (CFT) columns combined with wide flange (WF) steel beams in the design of moment resisting frames has many advantages. The success of this form of composite construction in seismic resistant design requires that the connections be capable of transferring forces among the adjacent members that are associated with inelastic response occurring in the beams. Currently, in the U.S. the knowledge and design guidelines to achieve this is lacking. The results of an ongoing research program involving the cyclic testing of full-scale structural connection subassemblies is presented, in which the columns and beams are CFTs and WF steel members, respectively. The objective of this multiphase research program is to assess the force transfer mechanism in these type of connections, examining the effect various structural details have on this mechanism, as well as on the connection's strength, stiffness, and ductility. Results from tests show that specimens in which the beams are designed to dissipate energy can have exceptional cyclic ductility. However, connections must be properly detailed to avoid strain concentrations which could lead to fracture. Measured deformations in the column show that a CFT column's virgin stiffness is well estimated by transformed section theory. However, interstory drift deformations beyond 0.5% of the story height tend to reduce the stiffness due to concrete cracking and debonding of the concrete from the steel tube.

KEYWORDS

concrete filled tube column, cyclic strength, diaphragm plates, displacement ductility, experimental behavior, local buckling, moment connection, panel zone, seismic resistance design

INTRODUCTION

The use of composite steel-concrete framing systems for buildings has increased over the past several decades, as designers have become more aware of the benefits of this form of construction. These benefits are derived from the attributes of each of the materials; namely, structural steel and concrete, and which includes the inherent mass, stiffness, damping, and economy of reinforced concrete, and the speed of construction, strength, long-span capability, and light weight of structural steel. There is generally two forms of composite construction for building frame systems. These include composite columns that are either structural steel shapes encased in reinforced concrete (SRC), or structural steel tubes filled with concrete. The latter is often referred to as a concrete filled tube (CFT), and will be the type of composite construction that will form the basis for this paper. The advantage of using a CFT composite column is that the steel tube provides permanent formwork and confinement for the concrete, thereby expediting construction. In addition, the concrete inhibits local buckling from occurring in the steel tube as well as acting as a heat sink for any extreme exterior temperatures that the tube may be exposed to in a fire.

One of the applications of CFT columns is to use them in conjunction with wide flange steel beams to form a perimeter moment resisting frame (MRF). The concrete that is placed in the tube provides local stiffening of the column as well as an increase in lateral stiffness which helps control drift of the frame. Under lateral seismic loading, the connections in CFT column MRF systems are subjected to large forces, resulting in the formulation of a large shear in the panel zone of a joint. The panel shear is the result of large beam flange forces that are transferred into the connection.

Currently in the U.S. there is a lack of seismic resistant design standards and general knowledge of the seismic performance of connections in CFT column-wide steel flange beam MRF systems. While research has been conducted in Japan, and which resulted in seismic design provisions (AIJ 1987), the differences in construction practices does not lend itself well to directly using these guidelines in the U.S. Further research based on large scale tests is therefore required that would provide information related to the seismic performance of details that the U.S. would utilize.

A research program was therefore planned and conducted at Lehigh University associated with the seismic performance of full-scale moment connection subassemblies of CFT column-to-wide flange steel beam systems [Vermaas et al. 1995]. The objective of this research was to examine the force transfer mechanism in the connection, and to evaluate the effects that connection details have on the cyclic strength, stiffness, and ductility. This information is intended to provide data and a basis for developing future recommendations for seismic resistant design. The research program consisted of several phases, and included both analytical and experimental studies. Reported herein are the results involving the initial testing of three full-scale connection subassemblies under seismic loading conditions. The specimens had interior and exterior diaphragms, respectively, in the connection region. The purpose of the diaphragms was to provide a local stiffening of the tube and a force path for the beam flange forces.

TEST MATRIX

The three connections are referenced as Specimens C1, C3, and C3R. Each specimen was comprised of a full moment connection between an interior CFT column and wide flange steel beams. Schematics of each of the three specimens are shown in Figure 1 through 3. For each specimen the beams were W24x62 sections and the column a steel tube of 406 x 406 x 12.5 mm dimensions. Specimen C1 had a set of interior diaphragms in the connection, with Specimens C3 and C3R having exterior diaphragms. The proportioning of the member sizes of the specimens corresponded to a full-scale model of the members in the ninth floor of a 20 story prototype perimeter CFT MRF. This prototype structure was an office building designed as a weak beam-strong column system for a seismic zone 4 in the U.S. The seismic forces were determined using the NEHRP provisions (NEHRP 1991), with the columns and steel beams designed using the ACI-89 ("American" 1989) and AISC LRFD ("American" 1994) provisions, respectively.

Measured dimensions and material properties for the beams and column for each specimen are given in Tables 1 and 2. Included in Tables 1 and 2 are the flexural capacities M_p of the W24x62 beams and the CFT column subjected to an axial gravity load P of 2000 kN. The column flexural strength M_p was computed per ACI criteria, using the measured column dimensions and material properties, where f'_c in Table 2 corresponds to the concrete compressive strength at the day of testing.

The connection elements, column, and panel zone were designed to resist the forces caused by a moment of $1.25M_p$ developing in both beams in order to ensure a weak beam-strong column system, in which primary yielding and energy dissipation occurred in the beams. The panel zone's shear capacity for each specimen was checked using AIJ provisions (AIJ 1987) to ensure that a shear failure in the panel zone did not occur.

The interior set of diaphragms of Specimen C1 were located opposite the beam flanges, using full penetration welds in conjunction with the self shielded flux core arc welding (FCAW) process to attach them to the inside of the steel tube (see Figure 1). The beam flanges were also welded to the column, using full penetration flux cored arc welding. The backing bars were subsequently removed, and the exposed weld back gouged and then profiled with a 6 mm fillet weld in order to improve the quality of the full penetration beam flange welds. A set of 89 mm long cover plates were subsequently welded to the beam flanges and column of Specimen C1, as shown in Figure 1. This was done in order that additional cover plates of greater length could be later added to strengthen the beams for future tests which would attempt to fail the panel zone. By pre-attaching the cover plates in this manner the additional cover plates could be attached to the beam without heating and thereby affecting the concrete in the steel tube. In Specimen C1, as well as the other test specimens, the shear tab was bolted to the beam web using five-A325 25 mm diameter bolts and then partially fillet welded. The placement of fillet welds along the shear tab, as shown in Figure 1, was required in accordance with AISC LRFD seismic design provisions ("American" 1994), since the plastic section modulus of the flanges accounted for less than 70% of the plastic moment capacity M_p of the beam. In addition, in all specimens six-102 mm long shear studs of 16 mm diameter were used to transfer each beam's gravity load to the concrete and provide resistance to prying under any moment developed in the shear tab.

Specimen C3 had a pair of exterior diaphragms fabricated from ST7.5x12 structural steel tees, which were attached to the beam flange and column as shown in Figure 2 using full penetration welds. The tees were of 241 mm length and designed to transfer to the steel tube within the panel zone the beams' tension flange force. Specimen C3R represented a retrofit of Specimen C3, following the testing of the latter, in order to enhance cyclic performance. This was accomplished by welding tapered, vertical steel plates of 230 mm length to each beam's flanges to create a condition of a more gradual transition of cross-section from the beam to the face of the column, as shown in Figure 3. This was necessary in order to reduce the magnitude of strain that became concentrated at the end of the connection detail of Specimen C3, and which caused fracture of the beam flanges.

TEST SETUP

In order that representative forces develop and boundary conditions exist in the vicinity of each connection, each specimen was tested as a structural assembly shown in Figure 4. The ends of the beams and column in the specimen structural subassembly corresponded to points of inflection at midspan and midheight in the prototype frame's beams and column when subjected to lateral loading. Therefore, the shear spans for a specimen's column and beams corresponded to the story height h and column spacing L of the prototype frame. The column of the test setup was axially loaded by a constant gravity load P using a system of two tension rods in conjunction with two hollow core actuators and a loading beam. A horizontally placed actuator was used to apply a lateral cyclic displacement history to the top of the column in order to develop overturning moments and forces in the specimen's members and connection corresponding to earthquake loading conditions. The lateral loading history for each specimen corresponded closely to the recommendation by ACT-24 (ATC 1992) for the cyclic seismic testing of structural steel components. A loading history basically consisted of initial sets of elastic cycles of progressively greater amplitude, followed by sets of inelastic cycles also of progressively greater amplitude. The amplitude of the inelastic displacement cycles were proportional to the displacement at first yielding of the specimen. During testing lateral bracing was used to prevent lateral, out-of-plane buckling of the specimen's beams and column.

The instrumentation plan for each specimen included: calibrated load cells to measure the applied lateral load H, gravity load P, and beam reactions developed in the rigid links; displacement transducers to measure the beam and column displacement at various locations along these members, as well as shear deformation in the connection's panel zone; rotation transducers to monitor the rotation at the ends of the beams and column, respectively; and, strain gages to measure the local strain in the beam flanges, connection diaphragm, and steel tube. Prior to testing, the steel members of a specimen were painted with whitewash in order to observe any signs of yielding by cracking and flaking of the applied coating.

TEST RESULTS

The beam moment-plastic rotation (M-θ_p) hysteretic response for all specimens is shown in Figures 5 through 7. Before testing Specimen C1, an accidental lateral overloading occurred, causing the specimen to develop inelastic deformation in its beams, in the east (push) direction of lateral column displacement. This inelastic deformation amounted to a residual displacement of 51 mm from the vertical plumb position at the top of the column. The displacement history of this specimen was therefore modified to include 6 initial elastic cycles before subjecting it to inelastic deformation in the west (pull) direction in order to straighten out the column to the vertical position. This required a total of about 100 mm of horizontal displacement to be imposed to the top of the column in the pull direction, in order to arrive at the plumb position. Unless otherwise noted, all response quantities in Specimen C1, including lateral displacement ductility μ and plastic beam rotation θ_p, are referenced from the column's plumb position.

The response of Specimens C1 and C3 both included a maximum displacement ductility of $\mu = 3$ before testing was stopped due to the onset of fracture in the beam flanges. In Specimen C1 this fracture occurred at the end of the 89 mm long cover plates, originating at the toe of the weld. The fracture in Specimen C3 occurred at the end of the structural tee. Strain gage readings and nonlinear finite element analysis indicated that these fractures were caused by a strain concentration. Because the cracks were stable, it was possible to continue to test to achieve a greater ductility in the specimens, however, this was at the risk of developing additional fracturing of the beam flanges, which may not be possible to repair and therefore not enable the specimens to be subsequently re-tested.

Prior to stopping the tests, the response of Specimens C1 and C3 were both exceptional. The beams of both specimens developed pronounced flange yielding over a length of about one beam depth from the end of the connection region. The steel tube within the panel zone region developed minor shear yielding, where a uniform shear strain of $\gamma = 0.004$ to 0.005 radians was measured from the strain gages placed across the panel zone. This amount of strain is equivalent to approximately two times the shear yield strain of the material. The maximum plastic rotation θ_{pmax} developed in the beams was 0.038 and 0.025 radians in Specimens C1 and C3, respectively, where the former is referenced from the column's initial starting position (see Figure 5)

The response of Specimen C3R had a lateral displacement ductility of $\mu = 5$, corresponding to an interstory drift ratio δ/h of 7.2%, before testing was stopped due to equipment limitations. The corresponding maximum plastic rotation θ_{pmax} developed in the east beam was 0.065 radians (see Figure 7). The west beam achieved a θ_{pmax} of 0.03 radians, corresponding to a subassembly drift δ of 125 mm, before a fracture occurred in the connection. This fracture originated along a weld between the corner of the steel tube and a structural tee's flange, causing eventually the fracture of both structural tees to occur at the bottom flange of the west beam. All other welds and parts of the connection remained intact in the two beams. An observation of the weld at the origin of fracture indicated the lack of placement of sufficient fillet weld behind the tee's flange during fabrication, which called for a 6 mm size fillet (see Detail 1 in Figure 2). The other tees all had 6 mm of fillet weld, which was successful in restraining cracking in the connection. The test was continued to achieve the displacement ductility of $\mu = 5$ by releasing the west beam's end reaction in order to deactivate it from the test setup.

Prior to fracture in Specimen C3R, both beams developed extensive yielding as well as initial local buckling in the flanges outside the connection region. The greater length of the connection in Specimen C3R caused larger moments to develop at the face of the column, where a plastic hinge formed in the beams at the end of the connection. The larger moments at the face of the column created a larger shear force in the panel zone compared to the other specimens, resulting in a panel zone shear deformation of about five times the shear yield strain. Following the release of the west beam, the east beam developed extensive local flange buckling as greater magnitudes of deformations were imposed to the test setup. The local buckling led to a gradual reduction in the beam's flexural capacity, as seen in Figure 7.

ASSESSMENT OF TEST RESULTS

The instrumentation of the specimens enabled the deformations of the various components of the specimen subassembly to be measured. From these measurements it was possible to determine the contributions of the column deformation δ_c to the interstory drift of the subassembly. The H-δ_c relationship was studied in an attempt to evaluate the flexural stiffness EI of the column. Shown in Figure 8(a) and (b) are the envelopes of the H-δ_c relationship for Specimens C3 and C3R corresponding to the first cycle of each amplitude of the displacement history. Included in these figures are elastic predictions of the column's deformation $\delta_{c,th}$, for a specified lateral load H, where

$$\delta_{c,th} = \frac{H(h - d_b)^3}{12EI} \tag{1}$$

For each specimen three predictions for $\delta_{c,th}$ were made by using EI for the column based on: (1) uncracked transformed section modulus EI_{tr}; (2) ACI 318-89 provisions for column stiffness EI_{ACI}; and, (3) the column stiffness EI_s considering only the steel tube to resist the deflections. The value for EI_{ACI} was based on

$$EI_{ACI} = \frac{E_c I_g}{5} + E_s I_s \tag{2}$$

where E_c, I_g, E_s, I_s are equal to the modulus of elasticity of the concrete, gross moment of inertia of the concrete core, modulus of elasticity of steel, and moment of inertia of the steel tube, respectively. The difference between EI_{tr} and EI_{ACI} was the use of the modular ratio of n = 5.9 instead of the value of five in Eqn (2).

An examination of Figure 8(a) indicates that the initial stiffness of the column of Specimen C3 is well predicted up to an interstory drift corresponding to $\delta = 0.005h$ when using the uncracked transformed section modulus. This was found to be also the case for Specimen C1. As the displacements increase, however, the column stiffness decreased due to concrete cracking and debonding of the steel tube from the concrete. In Specimen C3R it is apparent that the previous cyclic loading history, imposed on the specimen during testing as Specimen C3, had an effect on the initial elastic stiffness of Specimen C3R. The lateral stiffness of Specimen C3R is much closer to that of a specimen having a steel tube column with no filled concrete (e.g., EI_s). Hence, while the virgin elastic lateral stiffness of a CFT column appears to be well estimated using uncracked transformed section modulus theory, imposed lateral displacements beyond δ equal to 0.005h caused a permanent reduction in stiffness.

The proportioning of the connection for each specimen was based on a weak beam-strong column design (while also satisfying drift requirements in the prototype frame), where as noted previously the beam design moment for the connection was equal to 1.25 times the beam's flexural capacity (e.g. $1.25M_p$). The maximum beam moments M_{max} developed in the specimens at the end of the connection are shown in the M-θ_p beam relationship (see Figures 5, 6, and 7) to exceed their flexural capacity M_p that is based on the beam's measured dimensions and yield strength. The ratio of M_{max}/M_p for each specimen's beams are given in Table 1, where the value for M_{max} corresponds to the location at the end of the connection, and is shown to range from $1.13M_p$ (Specimen C3) to $1.23M_p$ (Specimens C1 and C3R). The assumption of designing for a maximum beam moment of $1.25M_p$ therefore appears reasonable (where these moments develop at the end of the connection) due to the fact that under significant cyclic loading strain hardening develops in the severely yielded beams. Specimen C3 had a smaller maximum beam moment of $1.13M_p$ compared to Specimens C1 and C3R's value of $1.23M_p$, since Specimen C3 was not subjected to as large of an amplitude of inelastic deformation, causing it to develop less strain hardening. The maximum plastic beam rotations $\theta_{p,max}$ of 0.038, 0.025, and 0.065 radians in Specimens C1, C3, and C3R, respectively, are of significant amount, particularly that which developed in Specimen C3R. Currently, in the aftermath of the recent 1994 Northridge earthquake, in which several welded wide flange steel beam-to-column moment connections fractured, the required amount of $\theta_{p,max}$ that a connection must process is being deliberated. It appears ["Interim 1995"] that for wide flange beam-to-column welded moment connections a required ductility (e.g., $\theta_{p,max}$) of 0.02 to 0.04 radians may be stipulated in future steel codes.

SUMMARY AND CONCLUSIONS

An ongoing research program on the subject of the seismic performance of moment connections for CFT column-to-wide flange steel beam MRF systems has been presented involving the cyclic testing of full-scale structural joint subassembly specimens. The test results indicated that connections with interior or exterior structural tee diaphragms have exceptional cyclic strength, stiffness, and ductility, where the beam is designed to yield in flexural and dissipate energy. The best performance is obtained when the details minimize the possibility of strain concentrations developing in the connection area, whereby beam flange fracture is inhibited. Under such conditions the maximum beam moment at the end of the connection (e.g., cover plates or exterior diaphragm, etc.) can approach 1.25 times its plastic flexural capacity. It was found that the initial elastic lateral stiffness of the CFT column was reasonably well estimated by using the uncracked transformed section. This holds however only for cyclic displacements that correspond to an interstory drift of less than 0.5% of the story height. More severe deformation caused cracking and debonding of the concrete from inside the tube, leading to a reduction in the elastic stiffness of the CFT column.

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REFERENCES

American Concrete Institute (1989). "Building Code Requirements for Reinforced Concrete (ACI 318-89) and Commentary-ACI 318R-89," Detroit, Michigan.

American Institute of Steel Construction (1994). "Load and Resistant Factored Design," 2nd Edition, *Manual of Steel Construction*, Chicago, Illinois.

Applied Technology Council (1992). "Guidelines for Cyclic Seismic Testing of Components of Steel Structures," Redwood City, CA.

Architectural Institute of Japan (1987). "Standard for Structural Calculation of Steel Reinforced Concrete Structures," Tokyo, Japan.

Graham, W.W., (1996). "Seismic Testing and Analysis of Full-Scale CFT-to-WF Beam Moment Welded and Bolted Connections," M.S. Thesis, Department of Civil and Environmental Engineering, Lehigh University, Bethlehem, PA.

"Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures," (1995). SAC Joint Venture, Report No. SAC-95-02.

National Earthquake Hazards Reduction Program - NEHRP (1991). "Recommended Provisions for the Development of Seismic Regulations for New Buildings," Building Seismic Safety Council, Federal Emergency Management Agency, Washington, D.C.

Vermaas, G.W., Ricles, J.M., and L.W. Lu, (1996). "Seismic Performance of Full-Scale Concrete Filled Steel Tube-to-Wide Flange Beam Welded Moment Connections with Diaphragms," ATLSS Engineering Center Research Report, Lehigh University, Bethlehem, PA.

Table 1 Beam Average Measured Dimensions and Material Properties, Maximum Experimental Beam Moments M_{max} and Plastic Rotation θ_{pmax}.

Spec					Flange		Web					
	b_f (mm)	d (mm)	t_f (mm)	t_w (mm)	σ_y (MPa)	σ_u (MPa)	σ_y (MPa)	σ_u (MPa)	M_p (kN-m)	M_{max} (kN-m)	$\frac{M_{max}}{M_p}$	$\theta_{p,max}$ (rad)
C1	179	603	14.9	10.9	293	456	342	475	777	954	1.23	0.038
C3	170	603	14.9	10.9	298	475	348	495	805	906	1.13	0.025
C3R	179	603	14.9	10.9	298	475	348	495	805	992	1.23	0.065

Table 2 Column Average Measured Dimensions and Material Properties,

Spec	b (mm)	D (mm)	t (mm)	b/t	σ_y (MPa)	σ_u (MPa)	f'_c (MPa)	M_p (kN-m)
C1	408	408	11.7	34.8	379	448	43.4	1242
C3	409	409	11.8	34.7	379	448	43.4	1245
C3R	409	409	11.8	34.7	379	448	53.4	1365

Note: σ_y and σ_u are mill test reported values.

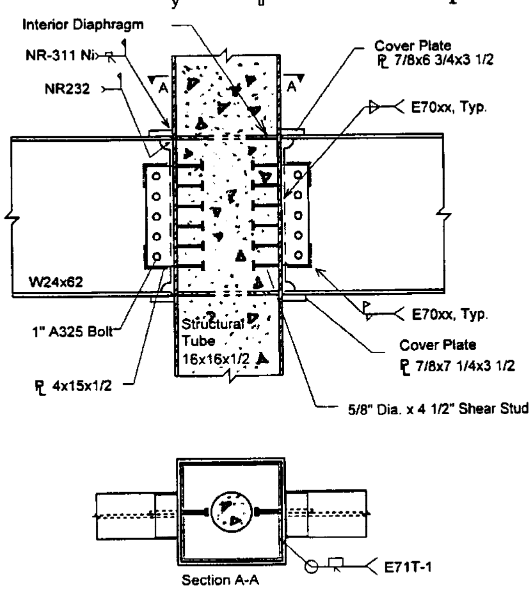

Fig. 1 Connection details, Specimen C1.

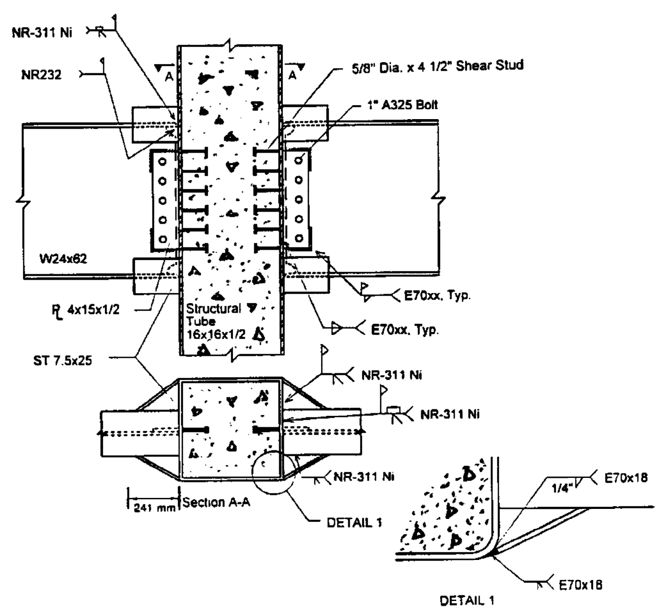

Fig. 2 Connection details, Specimen C3.

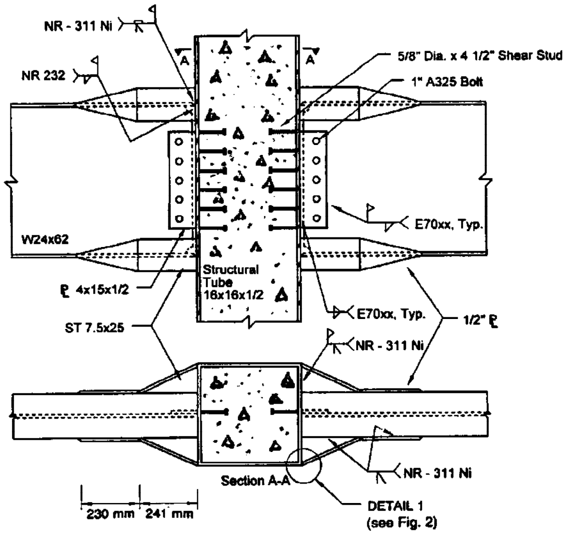

Fig. 3 Connection details, Specimen C3R.

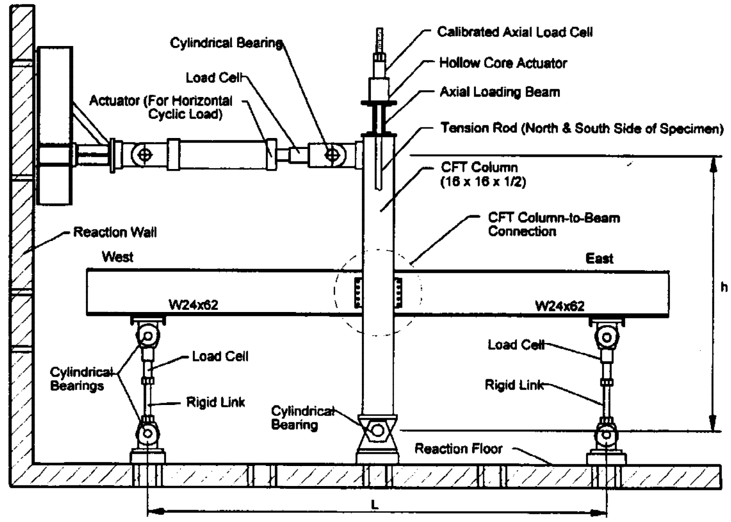

Fig. 4 Test setup.

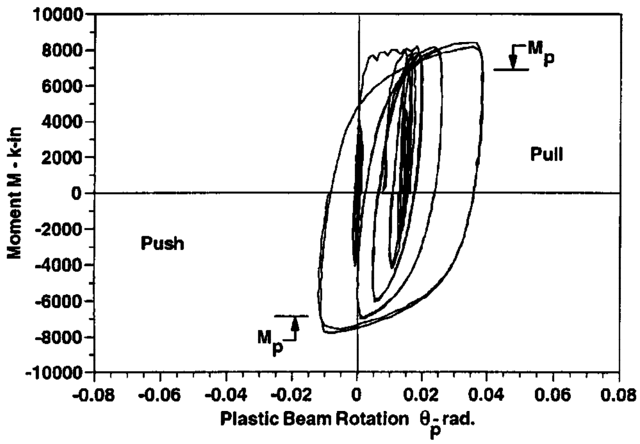

Fig. 5 Beam moment-plastic rotation response, Specimen C1.

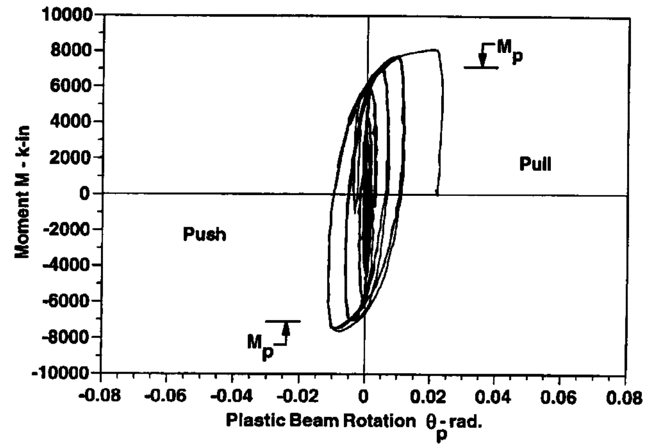

Fig. 6 Beam moment-plastic rotation response, Specimen C3.

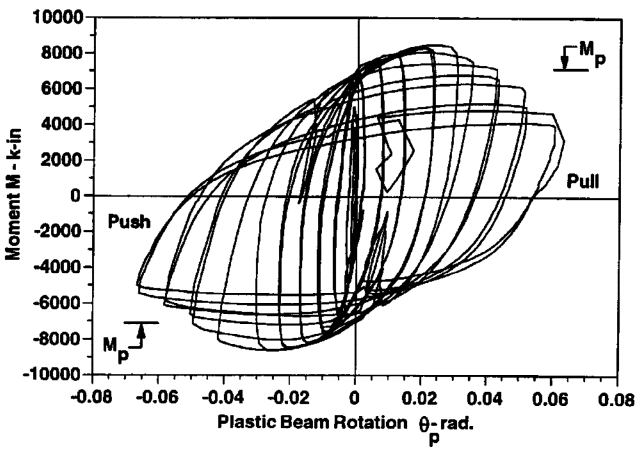

Fig. 7 Beam moment-plastic rotation response, Specimen C3R.

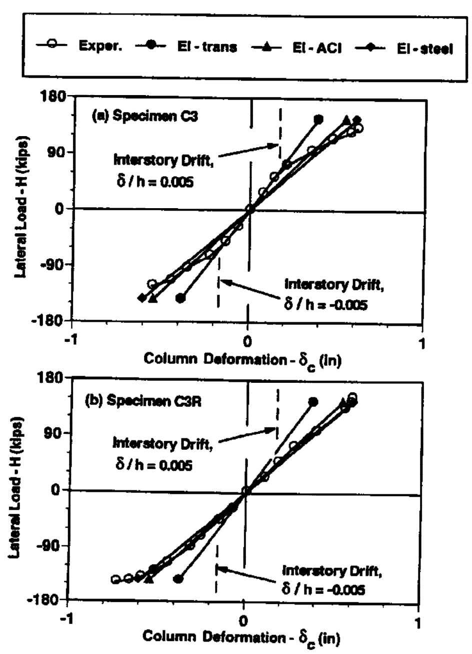

Fig. 8 Lateral load-column deformation response envelope, (a) Specimen C3, and (b) Specimen C3R.