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SEISMIC BEHAVIOR OF RCS BEAM-COLUMN-SLAB SUBASSEMBLIES DESIGNED FOLLOWING A CONNECTION DEFORMATION-BASED CAPACITY DESIGN APPROACH

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SUMMARY

This paper presents test results for four RCS subassemblies consisting of Reinforced Concrete (RC) columns and composite Steel (S) beams with reinforced concrete slab, under displacement reversals. The specimens were designed following a strong column-weak beam criterion, with the connection regions designed using a deformation-based capacity design procedure to limit joint deformations and damage. Two simple RCS joint details, consisting of either overlapping U-shaped stirrups passing through the steel beam web or steel band plates wrapping around the RC column just above and below the steel beam flanges were used to provide joint confinement. The performance of the specimens was evaluated in terms of lateral load vs. story drift response, beam and joint deformations, energy dissipation capacity, and story drift contributions from different structural components. Test results indicated satisfactory seismic performance of RCS subassemblies under large lateral displacement reversals. To evaluate the effect of joint deformations on RCS system behavior, dynamic analyses of a six-story RCS frame system under various ground motion records were conducted. In one of the three RCS moment-resisting frames analyzed, the composite joint regions were modeled as rigid joint panels. In the other two frames, the joint regions were modeled as flexible joint panels designed following a deformation-based design approach, which would limit the maximum joint shear deformations to approximately 1.2% and 0.5%. Results from inelastic dynamic analyses show that joint deformations may have a significant effect on maximum story drift, and thus joint flexibility shall not be neglected in analysis of RCS frames. In addition, different joint design philosophies may affect the inelastic deformation demand imposed on beams and columns.

INTRODUCTION

RCS moment-resisting frame systems, consisting of Reinforced Concrete (RC) columns and Steel (S) beams, take advantage of the inherent stiffness and damping, as well as low-cost of concrete, and the lightweight and construction efficiency of structural steel [1][2]. In the past thirty years, RCS moment-

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resisting frame systems have been mostly used for high-rise buildings located in regions of low seismicity. In recent years, research efforts have been made to develop seismic design guidelines for RCS systems located in regions of high seismic risk [3-6]. However, there was need for experimental evidence to evaluate the seismic behavior of RCS beam-column-slab subassemblies designed following a strong column/weak beam approach and using relatively simple and practical joint details.

This paper presents the results of a research investigation conducted at the University of Michigan that included the testing of four RCS beam-column-slab subassemblies under large lateral displacement reversals. These subassemblies were designed following a strong column/weak beam philosophy, and using a deformation-based approach in the design of the composite joints. The seismic performance of the RCS subassemblies and the efficiency of the deformation-based capacity design approach to limit joint deformations are evaluated in this paper. In addition, results from inelastic time-history analyses of a six-story RCS moment-resisting frame are presented. The effects of joint deformations on RCS frame behavior were investigated by modeling the joint regions as either rigid panels, or flexible panels designed for two different levels of maximum probable joint deformations.

EXPERIMENTAL PROGRAM

In the experimental investigation, four RCS beam-column-slab subassemblies were constructed and tested under reversed-cyclic loading at the University of Michigan Structural Engineering Laboratory. Each test specimen included RC columns and steel beams with an RC slab cast upon metal decks supported by the steel beams. The steel beams ran continuously through the column and full composite behavior between the RC slab and steel beams was intended through the use of shear studs. A brief description of the design philosophy used for the test specimens, and their primary design features, is given in the following.

Design Philosophy

The RC columns and composite beams were designed according to the ACI 318-99 Building Code [7], AISC-LRFD Code [8], and AISC Seismic Provisions [9]. The ACI strong column-weak beam design philosophy was followed when proportioning the beam and column sizes to ensure that most of the inelastic behavior within the RCS subassembly would occur in the beam regions adjacent to the RC columns. Because large joint deformations can lead to not only excessive story drifts, but also severe joint damage that would result in costly repairs after earthquakes, the composite joints were designed for 0.5% shear distortion so that the connections would experience only moderate damage during the tests.

To limit the level of joint deformation, and thus damage, a deformation-based RCS joint model developed by Parra and Wight, and Parra et al. [5][10] was followed. This model can predict with reasonable accuracy the shear force vs. shear deformation relationship for RCS joints with a variety of joint details, given that the joints are controlled by their shear strength rather than bearing strength [10]. In this experimental program, the joints were designed for a shear demand/shear capacity ratio leading to a maximum joint shear deformation of 0.5%. Based on test results from a previous study on RCS joints, this level of joint shear deformation would correspond to moderate concrete diagonal cracking in the joint region and slight yielding of the joint steel web panel. Joint shear demand is calculated assuming that the framing beams reach their ultimate moment strength. Joint shear capacity is calculated as the summation of three mechanisms: steel web panel, inner diagonal concrete struts, and outer diagonal concrete struts [5][11]. The shear resistance mechanism of the steel web panel in an RCS joint is similar to that in a structural steel frame. The inner diagonal concrete struts are located within the width of the steel beam flanges and can be activated through bearing of the concrete on the steel beam flanges and face bearing plates (FBPs) welded between the beam flanges at the column faces. The outer concrete struts act in the joint concrete outside the width of the steel beam flanges and can be mobilized through the use of shear keys, typically above and below the steel beam.

In this experimental program, joint bearing strength was checked according to the ASCE design guidelines for composite connections [11] to avoid possible bearing failure of the joint concrete above and below the steel beam flanges. At the 0.5% joint shear deformation level, a joint bearing deformation of similar magnitude can be expected [10], bringing the total joint deformation to approximately 1.0%.

Descriptions of Test Specimens

Each specimen consisted of a RC column, a steel beam that ran continuously through the column, and an RC slab cast on top of and connected to the steel beam to achieve full composite behavior through shear studs. Fig. 1 shows the test setup used in this investigation. In all test specimens, the RC column had a length of 2240 mm and a square cross section of 380 x 380 mm. Longitudinal column reinforcement included 12 No.19 deformed bars, representing approximately 2.4% of the column gross area. W12x16 steel shapes were used for the steel beams. The width-thickness ratio for the beam flanges and beam web was 7.5 and 49.4, respectively. The ratio for the beam flanges slightly exceeded the limiting value of 7.4 given in the AISC Seismic Provisions for preventing local buckling [9]. The beam properties were selected such that the maximum shear force that could be transferred to the joint when the beams reach their ultimate moment capacity would lead to a maximum joint shear distortion of approximately 0.5% in the interior subassemblies. As a unique feature for RCS construction, small steel sections (W6x16) were welded to the beam flanges and embedded in the RC columns to simulate the erection columns typically used in RCS construction.

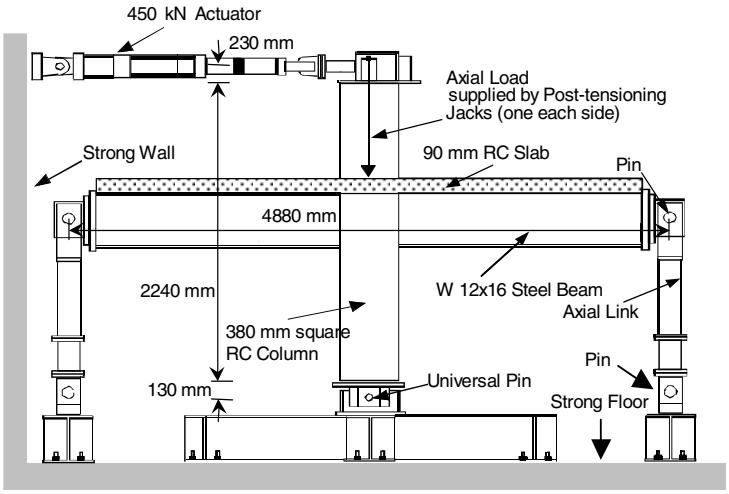
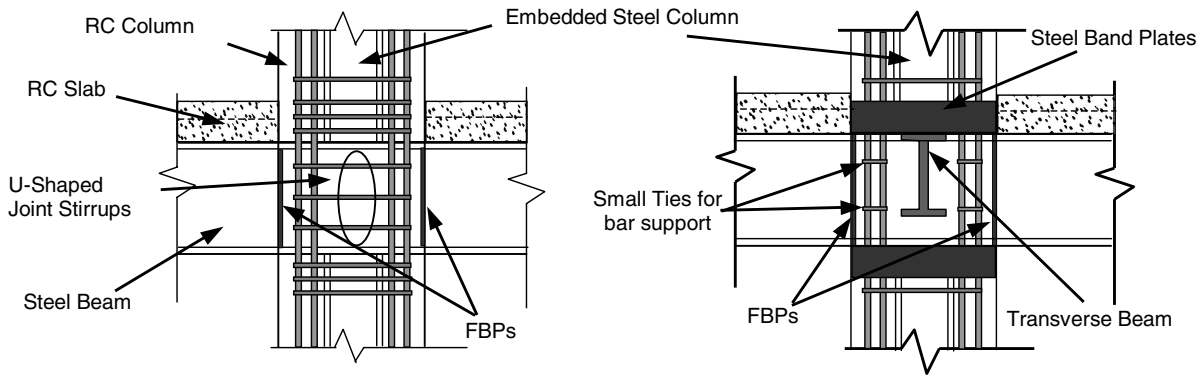


Fig. 1 Test Setup

In all four specimens, a galvanized 20-gage (1 mm) corrugated metal deck with a depth of 50 mm was placed on top of the steel beams, with the ribs parallel to the main beam. Shear studs with a length of 76 mm and a diameter of 20 mm were welded to the top beam flanges in a single line at 200 mm uniform spacing, with the first stud 25 mm away from the face of the column. A concrete slab with a total height of 90 mm was cast upon the metal deck. The width of the RC slab was selected to be 1220 mm. Slab reinforcement consisted of 4 No. 10 deformed bars in the direction parallel to the main beam, and No. 10 bars in the transverse direction at a uniform spacing of 300 mm. The RC slab was cast after the column was constructed, and thus the longitudinal slab bars did not pass through the RC column.

Two types of composite joint details were used in the experimental program. One of the details, shown in Fig. 2(a), consisted of overlapping U-shaped stirrups passing through holes drilled on the beam web. For this particular detail, transverse beams are assumed to frame into the main beam some distance away from

the connection to avoid joint construction difficulties. Also, closely spaced stirrups were placed in the column regions directly above and below the steel beam to provide confinement to these connection regions that are prone to fail by bearing, and to help mobilize the concrete regions outside the width of the steel beam flanges [11]. The other connection detail, shown in Fig. 2(b), featured steel band plates wrapping around the column regions just above and below the steel beams. In this detail, joint hoops that pass through the steel web panel were eliminated because of the excellent confinement provided by the steel band plates [4], allowing transverse beams to frame into the main beam at the connection region. To provide lateral support to the longitudinal column bars through the joint region, small ties that did not penetrate the steel web panel were provided through the joint depth (Fig. 2(b)). Each of the two joint details shown in Fig. 2 was used for one interior and one exterior subassembly in the test program.



(a) RCS joint with U-shaped stirrups (b) RCS joint with steel band plates

Fig. 2 RCS Joint Details Used in the Experimental Program

Specimen 1 was an interior subassembly and included overlapping U-shaped stirrups in the connection region. Three layers of No. 13 overlapping U-shaped stirrups were provided in the joint between the beam flanges, representing a stirrup volume-to-joint volume ratio of approximately 1.1%. FBPs, with a thickness of 13 mm and located at the front and back column faces, were fillet welded to the beam web and flanges to activate the inner diagonal concrete strut. In the column regions just above and below the steel beams, three layers of No. 10 stirrups, with two sets of closed rectangular hoops in each layer, were provided over a depth equal to 40% of the steel beam depth (d_{beam}), as recommended in the ASCE design guidelines [11]. The rest of the column was designed according to the ACI 318-99 [7] seismic provisions for RC columns.

Specimen 2 was an interior subassembly with steel band plates wrapped around the column just above and below the beam flanges, as shown in Fig. 2(b). These band plates were 13 mm thick and 76 mm high, connected by groove welding to form a closed box. The height of the band plates corresponded to 0.25 d_{beam}, a value recommended by Deierlein et al. [12] as the depth of the outer concrete panel beyond the steel beam depth. Internal stiffeners, oriented longitudinally above and below the beam web, were fillet welded to the band plates and the steel beam flanges. In addition, a fillet weld connection between the steel band plates and the beam flanges over the flange width was provided. The steel band plates in Specimen 2 replaced the column stirrups required just above and below the steel beam, and allowed the elimination of transverse reinforcement in the joint. This detail permits a direct connection of the transverse beams to the main beam at the joint region. The transverse beams used in Specimen 2 consisted of W8x13 steel sections and were shear connected to the web of the main beam through steel plates. Lateral support to the longitudinal column bars over the beam depth was provided through three layers of 6 mm diameter plain bars that did not penetrate the steel web panel (Fig. 2(b)). The other details of Specimen 2 were kept the same as for Specimen 1.

Specimens 3 and 4 had the same design as Specimens 1 and 2, respectively, except that they represented exterior subassemblies, and thus the composite beams framed into the column from only one side. Because a lower shear force would be transferred to the connection region in Specimens 3 and 4, joint shear distortions of approximately 0.3% were expected in the two exterior specimens. Detailed information about the test specimens can be found elsewhere [13].

Test Setup, Displacement History, and Instrumentation

The test setup used for this experimental program is shown in Fig. 1. All columns and beams were pin-connected at their ends to represent inflection points at member midspan. Lateral cyclic displacements were applied at the top of the column through a 450 kN hydraulic actuator. A small axial compression load equal to approximately 4.0% of the column axial capacity was applied to the columns through two hydraulic jacks, as shown in Fig. 1. A total of twenty lateral displacement cycles were applied to the specimens, with displacement amplitudes ranging from 0.5% up to 5.0% story drift. Each drift level cycle was repeated once to study the stiffness and strength loss at that drift level. Linear potentiometers and inclinometers were used to measure joint deformations and beam rotations. Strains in the steel beams and reinforcement in columns and slabs were monitored through linear and rosette strain gages. A load cell and an LVDT were used to monitor the applied lateral load and displacement at the top of the columns. A load cell was also attached to one of the steel links that supported the steel beams to monitor beam shear. A potentiometer was also placed near the bottom of the column to monitor the slip of the test setup during the tests. The drift levels described in the following section have been adjusted to account for this slip.

EXPERIMENTAL RESULTS

Joint Concrete Cracking Pattern and Behavior of Composite Beams

For the two interior subassemblies (Specimens 1 and 2), diagonal cracks first occurred in the joint region at approximately 0.5% story drift. For Specimen 2, which had transverse beams, diagonal cracks originated from the tips of the bottom flange of the transverse beams. Flexural cracks across the width of the concrete slab were also observed during the cycle to 0.5% drift. Beam yielding was first observed at 1.0% story drift. At 2.0% drift, after significant beam yielding had taken place, local buckling was observed in the beam bottom flanges and webs. Both Specimens 1 and 2 reached their maximum lateral load capacity during the first cycle to 3.0% drift, and most of the joint cracking occurred before that drift level. After 3.0% drift, inelastic behavior concentrated in the beam plastic hinge regions near the column faces. At the end of the tests, only moderate damage was observed in the joint region of Specimens 1 and 2, as intended in design. Due to the presence of the concrete slab, local buckling was prevented in the beam top flanges in Specimens 1 and 2 throughout the tests. Some crushing of slab concrete near the column faces occurred shortly after the specimens reached their maximum lateral strength. No apparent damage was observed in the shear studs in the beam plastic hinge regions during the tests. Joint cracking and beam yielding at the end of the test in Specimen 1 are shown in Fig. 3(a).

For the two exterior beam-column-slab subassemblies, Specimens 3 and 4, only slight joint damage occurred due to lower shear forces transferred into the connection compared to the interior subassemblies. For these two specimens, hairline diagonal cracks started to form at approximately 0.5% drift in the joint region, and a few new diagonal cracks formed during the cycles to 1.5% and 2.0% drift. No significant cracking occurred in the joint region for larger drift levels because of beam yielding, which limited the amount of shear force transferred into the joint. In Specimens 3 and 4, beam yielding was first observed at 1.5% drift. At 3.0% drift significant local buckling occurred in the beam bottom flanges and webs. Crushing of slab concrete was also observed in the two exterior specimens after they reached their

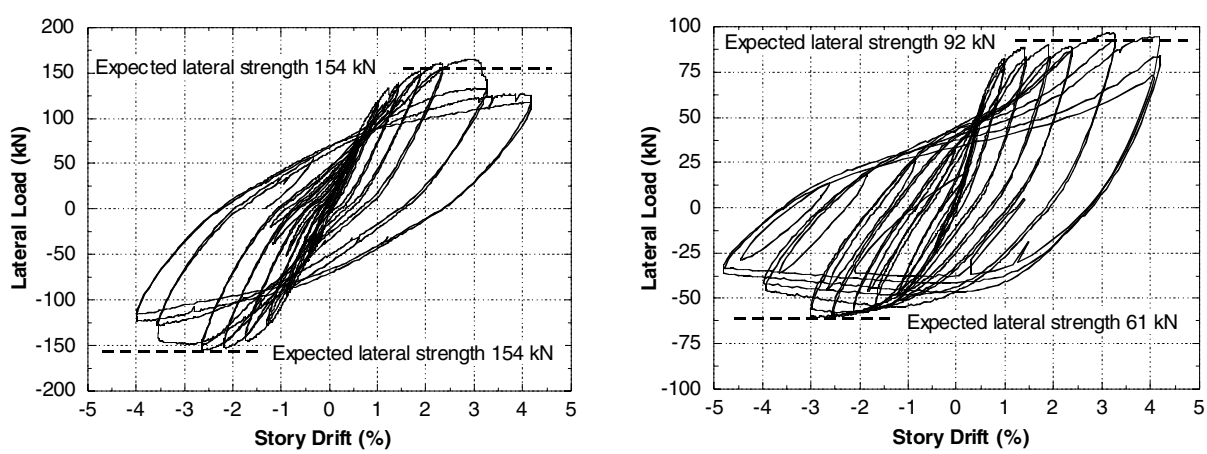
maximum lateral strengths. The joint cracking pattern and beam local buckling in Specimen 4 are shown in Fig. 3(b).

(a) Specimen 1 (b) Specimen 4

Fig. 3 Joint Cracking and Beam Yielding in Test Specimens at End of Tests

Lateral Load vs. Story Drift

The lateral load vs. story drift response for Specimens 1 and 4, which represent a typical response for the interior and exterior subassemblies, respectively, are shown in Figs. 4 (a) and (b). The total drifts were corrected to account for deformations in the test setup. For the interior specimens, slight pinching can be noticed in the hysteresis loops for the cycles below 3.0% drift, primarily due to joint diagonal cracking and bearing deformations (Fig. 4(a)). At larger displacement cycles, during which large beam flexural deformations occurred, full hysteresis loops that led to excellent energy dissipation capacity were observed. Both interior specimens exhibited stable responses, retaining more than 75% of their peak strength at the end of the tests. For the exterior specimens, positive displacement corresponded to positive bending (slab in compression) in the beam. The load vs. story drift response was not symmetrical due to the presence of the concrete slab, as shown in Fig. 4(b). These specimens were stronger for the positive loading direction, for which the concrete slab was engaged in compression. Because of the minor damage in the joint region, the overall response of Specimens 3 and 4 was dominated by beam rotations. The drops in lateral strength in all the specimens after they reached maximum lateral capacity, as shown in Figs. 4 (a) and (b), were mainly caused by local buckling of the beam bottom flanges and webs.



(a) Specimen 1 (b) Specimen 4

Fig. 4 Lateral Load vs. Story Drift Response

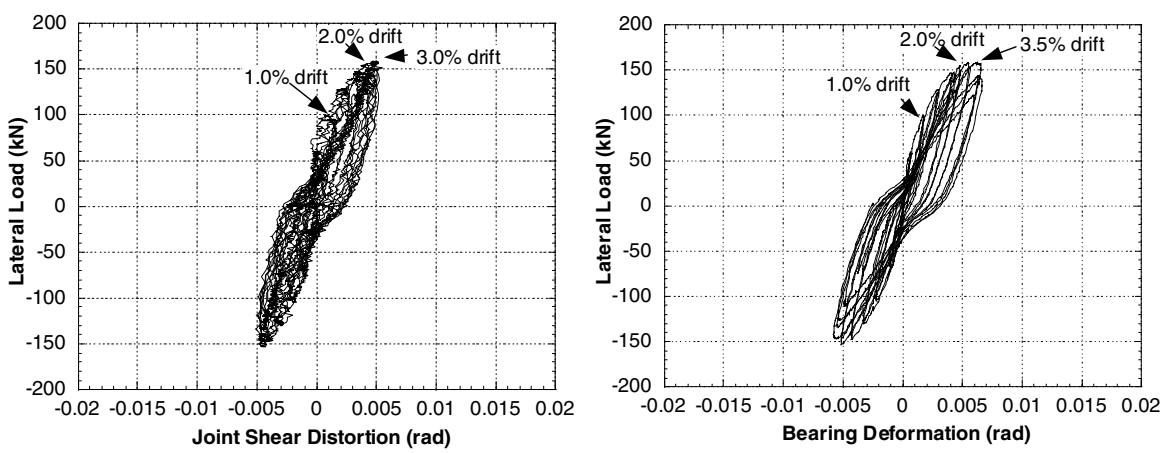
Beam Rotations

In the test specimens, beam rotations were measured using linear displacement transducers over a length equal to 75% of the total depth of the composite beam from the column face. For the interior specimens, total beam rotations larger than 0.03 rad were measured in both positive and negative bending directions. For the exterior specimens, maximum beam rotations of 0.03 rad were measured in the positive bending direction. In the negative direction, maximum beam rotations of 0.05 rad were measured at the end of the tests.

Predicted vs. Measured Joint Deformations

Total joint deformation in RCS joints includes joint shear deformation and joint bearing deformation. Joint shear deformation is caused by the shear force transferred into the joint from adjacent beams and columns. Joint bearing deformation represents the rigid body rotation of the steel beam within the column caused by the bearing of steel beam flanges against the surrounding column concrete. Figs. 5(a) and (b) show the measured joint shear and bearing deformation responses, respectively, for Specimen 2. As indicated in Fig. 5(a), a maximum joint shear deformation of 0.005 rad was measured at 3.0% story drift, when the specimen reached its maximum lateral strength. Joint bearing deformation response is shown in Fig. 5(b), with a maximum value of approximately 0.006 rad. The peak total joint deformation (summation of joint shear and bearing deformation) for Specimen 2, was approximately 0.011 rad. Joint response of Specimen 1 was similar to that of Specimen 2, with measured joint shear, bearing, and total deformations of approximately 0.006, 0.008, and 0.014 rad, respectively. The joint deformations in the exterior specimens were smaller compared to those of interior specimens, as a result of the relatively low shear demand imposed on the exterior joints. In Specimens 3 and 4, measured maximum joint shear and bearing deformations were 0.003 and 0.006 rad, respectively.

As mentioned earlier, the target joint shear deformation for the interior specimens was 0.005 rad. Fig. 6 shows a comparison between the measured joint shear deformation envelope for Specimen 1 and the predicted response using the deformation-based joint model [5]. As can be seen, the predicted response agreed well with the test results, indicating that the joint model can be used effectively for deformation-based capacity design of RCS composite joints in regions of high seismic risk.



(a) Shear Deformation (b) Bearing Deformation

Fig. 5 Lateral Load vs. Joint Deformation Response of Specimen 2

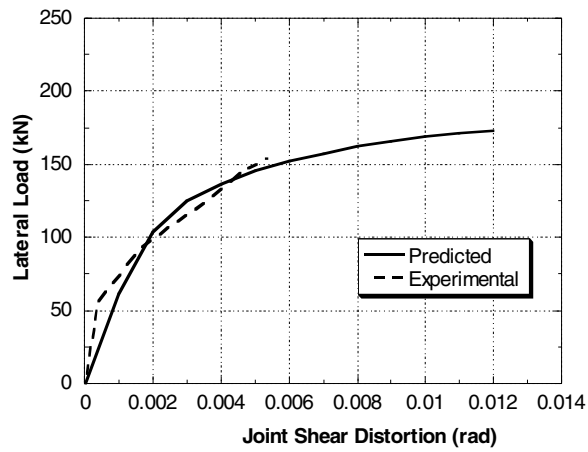


Fig. 6 Comparison between Measured Joint Shear Deformation Envelope and Predicted Response (Specimen 1)

Drift Components

Four major mechanisms contributed to the total story drift in the test subassemblies: 1) beam rotations, including elastic and plastic rotations, 2) column rotations, 3) joint shear distortions, and 4) joint bearing deformations. Fig. 7 shows typical contributions from these four mechanisms for an interior subassembly. The calculated drift values were all within 8% of the actual story drift, as can be observed in the figure. At 2.0% story drift, total joint deformation contributed to approximately 40% of the total story drift for the interior specimens, while beam and column rotations contributed to 50% and 10% of total story drift, respectively. At 5.0% story drift, the beam contribution increased to 75% of the total story drift, while the joint contribution reduced to 20%. For the exterior specimens, joint deformations contributed approximately 15% of total story drift at 2.0% story drift and this contribution decreased to less than 5% at 5.0% story drift. Beam rotations dominated the drift response of the exterior subassemblies.

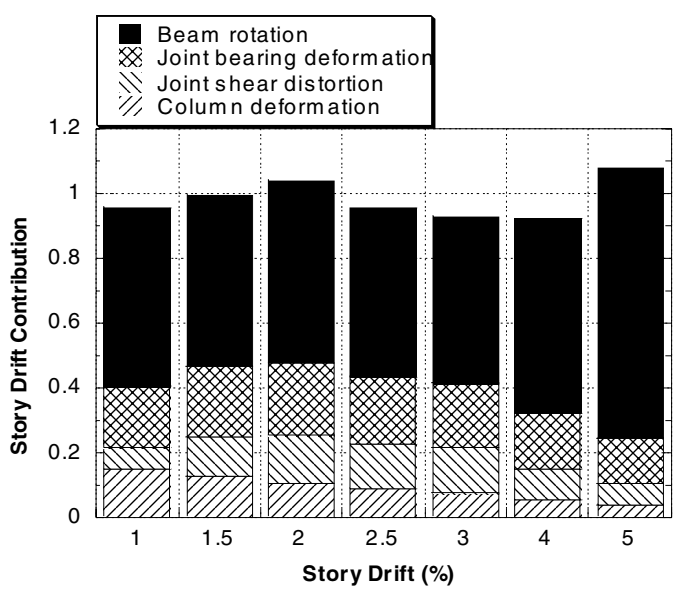


Fig. 7 Drift Components (Specimen 1)

EFFECTS OF JOINT DEFORMATIONS ON SYSTEM BEHAVIOR

Typically, connection regions are modeled as rigid zones in analysis of RC frame structures. This practice might be adequate if negligible deformations are expected in beam-column connections. However, test results in this experimental program have shown that joint deformations in RCS subassemblies designed following a strong column-weak beam philosophy could contribute up to 40% of total story drift (Fig. 7), and thus their influence on frame response should not be ignored. In order to evaluate the influence of joint deformations on the seismic behavior of RCS frame systems, a series of inelastic dynamic analyses were conducted on three six-story RCS frames. In one of the frames, the joint regions were modeled as rigid zones. The other two frames used flexible panel zone elements to account for joint deformations. The joint panels in these two frames were designed using either a strength-based or a deformation-based joint design procedure, which would limit the maximum probable joint shear deformation to approximately 1.2% and 0.5%, respectively.

Strength-Based and Deformation-Based Joint Capacity Design

For the design of an RCS joint, its shear strength needs to be checked against the shear demand as follows [14],

$$\phi k V_{uj} \geq V_d \qquad [1]$$

In Eq. [1], V_{uj} is the ultimate joint shear strength, typically taken as the strength at 1.2% joint shear distortion. The joint shear demand, V_d, should be calculated assuming that the beams framing into the connection in the loading direction reach their ultimate moment capacity, taking into account material overstrength and strain hardening in the steel sections of the composite beams. The value for the factor k depends on the target maximum level of joint shear distortion when framing beams reach their peak capacity. As shown in Fig. 8, in joints designed following a deformation-based procedure, k = 0.85 is used, as opposed to k = 1.0 in the strength-based design method. When using Eq. [1], material overstrength and strain hardening of the steel joint web panel are not accounted for in the calculation of joint shear strength, V_{uj}, while these effects are considered for the steel beams when determining the joint shear demand, V_d. Thus, an additional strength reduction factor is implied in Eq. [1]. In this analytical study, this effect was included in the joint panel modeling, as explained in the following section. In both strength-based and deformation-based joint designs, a ϕ factor equal to 1.0 was used.

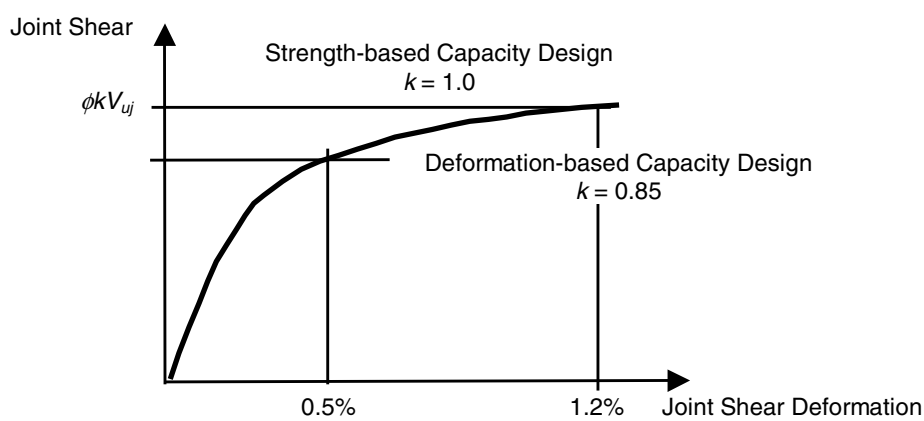
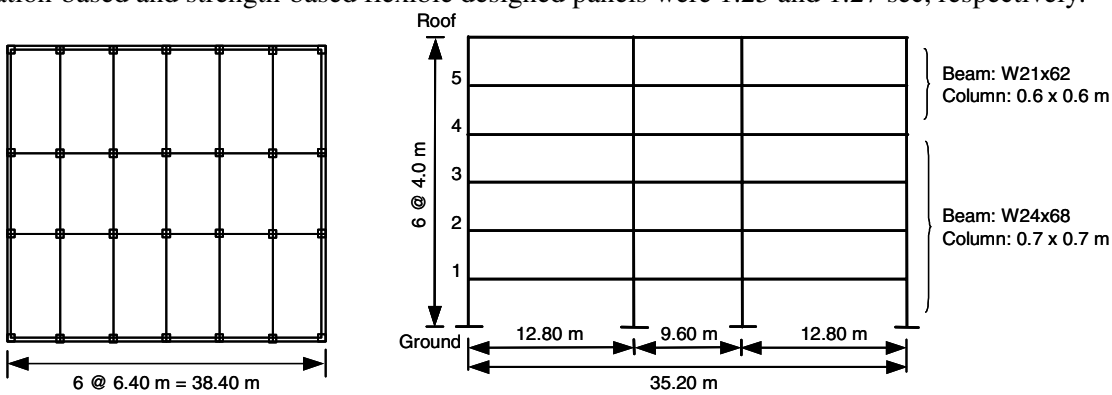


Fig. 8 Strength-Based vs. Deformation-Based Joint Capacity Design Methods

Inelastic Time-History Analysis

The studied RCS frames were selected from a prototype six-story building developed during the US-Japan Cooperative Earthquake Engineering Research Program on Hybrid and Composite Structures. The plan and elevation view of the building is shown in Fig. 9. The seismic design of the buildings followed the 2000 IBC Code [15], assuming the buildings were located in the Los Angeles area, site class D (stiff soil), and assigned to Seismic Use Group I and Seismic Design Category D. The mapped 5% damping spectral accelerations at 0.2 sec and 1.0 sec periods for the maximum considered earthquake at the site were S_s = 1.5g and S_1 = 0.72g, respectively. The seismic weight assigned to the frame for the typical floor was 1063 kN, and 858 kN for the roof. The final beam and column design was similar to that from an analytical study conducted by Mehanny [6], but with modifications to column dimensions and reinforcement. The building design had no correlation with the test subassemblies described earlier. The fundamental period for the frame with rigid joint panels was 1.03 sec, and the fundamental periods for the frames with deformation-based and strength-based flexible designed panels were 1.25 and 1.27 sec, respectively.



(a) Plan view (b) Elevation view

Fig. 9 Six-Story RCS Frame Building Used in Dynamic Analyses

The computer program RAM Perform-2D [16] was used to study the dynamic response of the RCS frames. Element properties for the beams, columns and connection panels were first calibrated with experimental results from this and a previous study on exterior RCS joints at the University of Michigan. An inelastic panel zone element was used to model joint deformations in the two RCS frames that considered joint flexibility. Beams were modeled as an elastic segment with inelastic hinges at the face of the columns and rigid ends within the column depth. Columns were modeled as an elastic segment with inelastic hinges at their ends.

Because the purpose of the analytical study was to investigate the effect of joint deformation on the overall behavior of RCS frames, the shear strength of the composite joints in the studied frames, V_{uj}, was determined based on the joint shear demand, V_d, divided by ϕ and the appropriate k factor, rather than from specific joint details and designs. For joint modeling purposes, a 10% increase in joint shear strength was considered due to material overstrength and strain hardening in the steel joint web panel. A simplified tri-linear joint shear force vs. total joint deformation relationship was used for the composite joints in the dynamic analyses. It should be mentioned that the panel element in RAM Perform-2D was developed to model joint shear distortions only. As a reasonable simplified approach, joint bearing deformations were combined with shear distortions in the same panel element for the dynamic analyses.

Four ground motion records representing different characteristics and intensities were used for the dynamic analyses: two El Centro records and two Sylmar records from the Northridge earthquake. The El Centro records were selected to represent far field ground motions, while the Sylmar records were selected because of their near-fault characteristics. The two records from the same earthquake were scaled to represent a probability of exceedance of 10%/50 years and 2%/50 years, respectively, for the Los Angeles area. The El Centro 10%/50 years record and the two Sylmar records were directly selected from the suite of ground motions developed for the SAC project by Somerville et al. [17]. The fourth record, the El Centro 2%/50 years record, was scaled by the authors based on the El Centro 10%/50 years record from SAC. A scaling factor that provides a match between the 5% damping response spectrum for the scaled record and the NEHRP response spectrum at the average fundamental period (approximately 1.26 sec) of the frames with flexible joint panels was used.

Effect of Joint Deformation on Interstory Drift

The maximum interstory drifts in the three frames under the selected earthquake records are shown in Fig. 10. As can be observed in this figure, when the RCS connections were modeled as rigid zones, the maximum interstory drift under the 10%/50 year Sylmar event was 1.5%. However, when flexible joint panel zones were included in the model, the maximum interstory drift increased to approximately 2.1%. Similarly, the maximum interstory drift increased from 2.3% to approximately 2.9% under the 2%/50 year El Centro record, and from 2.5% to 3.3% under the 2%/50 year Sylmar record when connection flexibility was considered in the analyses. For the 10%/50 year El Centro record, the maximum interstory drifts for the three frame models were approximately the same, mainly because joint shear demands were low, and thus negligible deformations took place in the RCS connections.

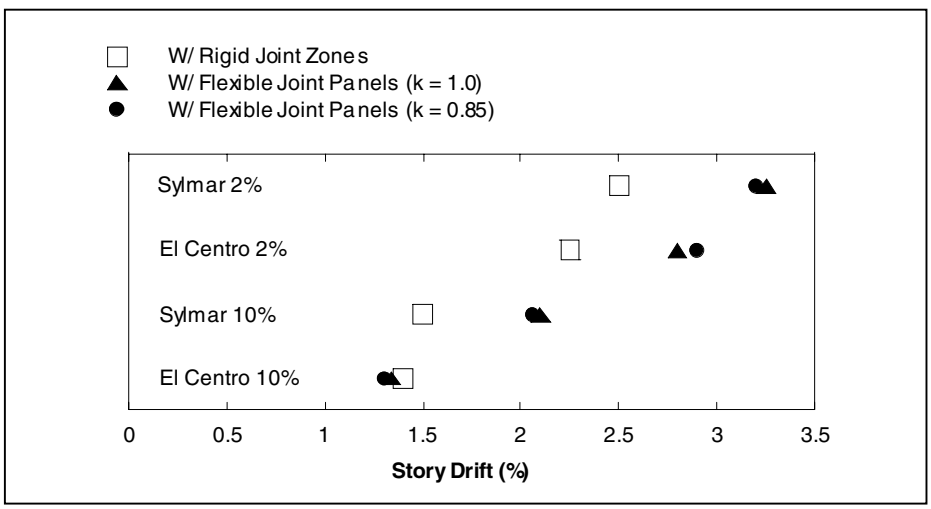


Fig. 10 Comparison of Maximum Interstory Story Drifts

Effect of Joint Design Philosophy on Joint Deformation Demand

A comparison of the maximum joint deformations experienced by the two frames with joint panels is shown in Fig. 11. For the frame with joints designed following a strength-based approach, the maximum total joint deformation was 1.2%, which corresponds to moderate joint damage as observed in the subassembly tests. For the frames with joints designed for controlled deformation, the maximum joint distortions decreased to 0.8%, which is approximately equal to the joint elastic deformation limit. The relatively low deformations obtained in the joints can be attributed to the fact that the beams framing into the two sides of the connections did not simultaneously reach their ultimate moment strength. As a result, the shear force transferred to the connection was lower than the extreme case considered in Eq. [1]. Analytical results also indicated that the use of different joint design philosophies resulted in changes in

the distribution of inelastic behavior among the members. A deformation-based joint design limited the level of joint deformations, while forcing slightly larger beam rotations in the frame.

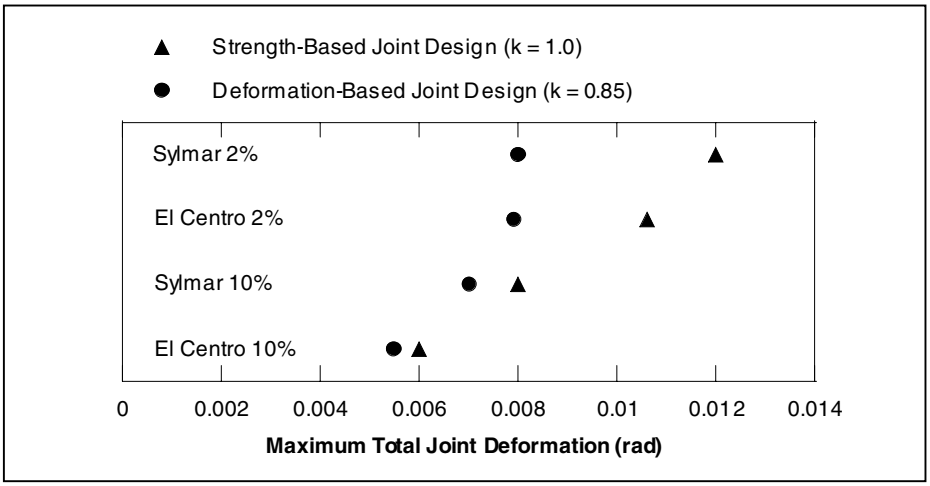


Fig. 11 Comparison of Maximum Total Joint Deformations

SUMMARY AND CONCLUSIONS

Four RCS beam-column-slab subassemblies, two interior and two exterior, were tested under large displacement reversals. The specimens were designed following a strong column-weak beam philosophy and a joint deformation-based capacity design procedure to control connection distortions and damage. The behavior of the RCS subassemblies was evaluated in terms of load vs. displacement response, joint deformations, beam rotations, and contribution from different deformation mechanisms to story drift. To complement the experimental program, a series of inelastic dynamic analyses were performed on a 6-story RCS frame to study the influence of joint deformations and design philosophy on the overall behavior of RCS frame systems.

The four RCS beam-column-slab subassemblies tested in this investigation showed good seismic performance with stable load vs. story drift response, excellent energy dissipation, large beam rotations, and only minor to moderate joint damage. A comparison between target and maximum joint shear deformations indicated that the use of a deformation-based capacity design procedure for RCS connections was effective in controlling connection distortions and damage. From the analytical investigation, it was found that joint deformations in RCS frames could lead to a significant increase in maximum interstory drift, and thus connection flexibility should be considered in the analyses of these hybrid systems. The use of a deformation-based joint design method led to smaller joint deformation demands compared to those in the frame with connections designed following a strength-based approach.

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