BEHAVIOR OF RCS ROOF T-CONNECTIONS UNDER LOAD REVERSALS

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SUMMARY

Results from the test of one hybrid Reinforced Concrete (RC) column-to-Steel (S) beam (RCS) T-connection subassembly subjected to load reversals are presented. The joint configuration included Face Bearing Plates (FBPs), Steel Band Plates (SBPs) wrapping around the RC column just above and below the steel beam, and vertical U-shaped stirrups. Longitudinal column bars were terminated with mechanical anchors in order to reduce reinforcement congestion and improve joint bearing resistance. The test specimen exhibited a stable response up to the first cycle to 4.0% drift, with moderately pinched hysteresis loops. During subsequent cycles at 4.0% and 5.0% drift, a significant decay in strength occurred due to severe slip of column longitudinal bars. The behavior of the subassembly is described in terms of lateral load versus displacement response, joint distortions, stiffness retention, and energy dissipation capacity.

Keywords: composite structure, RCS joint, shear distortion, shear strength, bearing deformation.

INTRODUCTION

A relatively new type of structural system is represented by hybrid RCS frames that consist of Reinforced Concrete (RC) columns and Steel (S) beams passing continuously through the column. This system has gained popularity among the structural engineering community in the past twenty years because of its economical advantages and excellent performance, particularly when subjected to earthquake type loading. RC columns have been shown to be significantly more cost-effective than steel columns (Griffis 1986) and provide excellent stiffness to the structure, while connections between RC columns and continuous steel beams eliminate the need for welding, and thus the potential risk of premature joint fracture. Steel floor systems are lighter than conventional RC beam-slab systems, leading to significant reductions in building weight and foundation costs (Griffis 1986).

During the last two decades, results from experimental programs on RCS interior and exterior joints conducted in the U.S. and Japan have demonstrated the potential of RCS frame structures for use in earthquake prone areas. Despite the promising future of these hybrid structures, little is known about the behavior of roof T-connections, in which two beams and only one column frame into the joint. Although the behavior of roof connections might not significantly affect the system response in medium and high rise buildings during ground shakings, the response of roof connections may have an important influence on the seismic behavior of low rise structures. Therefore, an experimental and analytical program was recently undertaken at the University of Michigan to study the inelastic cyclic response of RCS roof T-connections.

In this paper, preliminary results from the test of one RCS roof T-connection subassembly are discussed in terms of load versus displacement response, joint distortions, stiffness retention, and energy dissipation capacity. This subassembly is the first specimen tested in the experimental program. Currently, the second specimen is under construction and it is expected to be tested in October 2003.

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**SPECIMEN DESCRIPTION**

The beam-column connection in the test specimen was designed using the shear strength equations proposed by Parra and Wight for exterior joints (Parra et al. 2001). Although these equations might not be appropriate for roof T-connections, the writers believe that the shear strength of those joints should not differ significantly from that of exterior joints, given that bearing failures are prevented. The ratio between the summation of nominal beam plastic moment capacities to nominal moment strength of the concrete column was 1.2 and the connection represented the weakest link of the subassembly that was tested.

The test specimen consisted of a 16 in. x 16 in. RC column and a W8x28 steel beam passing continuously through the column. A W6x25 steel section was embedded in the RC column to simulate steel columns typically used during the frame erection process. Column longitudinal reinforcement consisted of 12 #6 bars, corresponding to 2.1% of the gross concrete area. Headed bars (Fig. 1) were used to avoid the need for hooks above the top steel beam flange, which would not be effective in contributing to joint bearing strength while also leading to reinforcement congestion. The head area was four times that of the bar and was terminated 2 in. above the top flange. To simulate the effect of the portion of the slab just above the connection, a 4 in. concrete topping was cast on top of the joint region above the steel beam flange, as shown in Fig. 1. Column transverse reinforcement consisted of 3 – 4 leg stirrups at 4 in. spacing. Other joint details included ½ in. thick Face Bearing Plates (FBPs) welded to the beam web and flanges at the column faces to enhance participation of concrete within the beam flanges. Steel Band Plates (SBPs), ½ in. thick and wrapping around the column regions just above and below the steel beam, were used to provide confinement to the connection. In addition, four #3 vertical U-shaped stirrups, which were placed across the top flange of the beam and extended through the joint, were used to help resisting the upward thrust caused by bearing of the top flange on the concrete slab. A sketch of the connection details is shown in Fig. 1.

**MATERIAL PROPERTIES**

Dual Grade A36/A572-Grade 50 steel was used for the steel beam and the embedded steel column. Ready-mix concrete from a local supplier was used for the RC column. The average compressive strength of this concrete, obtained from cylinder tests, was 4.3 ksi. The FBPs and SBPs were made of A36 steel. Grade 60 steel was used for the RC column longitudinal bars, transverse reinforcement and vertical U-shaped stirrups.

**TEST SETUP, LOADING HISTORY AND INSTRUMENTATION**

A sketch of the test setup used in this investigation is shown in Fig. 2. The specimen was rotated 90 degrees from its original position in order to accommodate an existing test setup in the University of Michigan Structural Engineering Laboratory. The steel beam and RC column were pinned at mid length to simulate inflection points, and the load was applied at one beam end through a 100-kip hydraulic actuator. A load cell and LVDT were used to monitor the applied load and lateral displacement, respectively.
A total of fifteen lateral displacement cycles were applied to the specimen, ranging from 0.5% to 5.0% drift, as shown in Fig. 3. Rosette and linear gages were used to measure strains in the steel beam and column reinforcement. Linear potentiometers were utilized to monitor column and beam rotations, as well as joint distortions.

EXPERIMENTAL RESULTS

Cracking Pattern and Lateral Load versus Story Drift Response

Joint diagonal cracks were first observed at 1.0% drift, accompanied by flexural cracks in the column region adjacent to the connection. As the test progressed, these diagonal cracks started to propagate in both loading directions and at 2.0% drift several diagonal cracks had already crossed the joint region. At 3.0% drift, the connection was visibly damaged (Fig. 4a). In addition, column reinforcement slip through the connection led to the opening of a gap between the RC column and the steel beam (Fig. 4b), and thus to softening of the subassembly. At 5.0% drift, the connection was severely damaged (Fig. 4c), with crack widths exceeding 0.12 in. and crushing of concrete in the regions adjacent to the headed bars (Fig. 4d), which allowed severe slippage of column longitudinal reinforcement.
The load versus displacement behavior of the subassembly is shown in Fig. 5. Up to 1.0% drift, the specimen behaved in the cracked elastic range and reached approximately 50% of its maximum lateral strength. From 1.5% drift to the first drift cycle at 4.0% drift, the specimen exhibited stable behavior with only a small decay in lateral resistance. The peak lateral force (23.0 kips) was attained during the first cycle to 3.0% drift, decaying to 18.9 kips during the first cycle to 4.0% drift. For the second cycle to 4.0% drift, a substantial decrease in specimen strength was observed, mainly due to excessive bearing distortions caused by reinforcement slip. The strength of the RCS roof T-connection appeared to be lower than that predicted for an exterior joint. Using the design equations for exterior connections proposed by Parra and Wight (Parra et al. 2001), the predicted nominal shear strength for the connection corresponded to a lateral load of 26.8 kips, compared to a maximum of 23.0 kips exhibited by the test specimen. Even though the cracking pattern suggests severe damage due to high shear distortions, the predominant specimen failure mode was a bearing failure.
Deformation of the Composite Connection

In composite RCS joints, two types of deformations have been identified (ASCE, 1994; Deierlein et al., 1989), Joint Shear Distortion (JSD), which is similar to that observed in RC and steel connections, and bearing distortion due to high bearing stresses in the concrete regions adjacent to the steel beam flanges. The summation of JSD and bearing distortion is typically referred to as total joint distortion in RCS connections.

The JSD in the test specimen was measured through linear potentiometers located on one face of the connection and through rosette gages placed on the steel web panel. The JSD measured through linear potentiometers, however, was not reliable because of problems encountered with one of the potentiometers during the test. Therefore, the JSD response presented herein is that monitored by a rosette gage located in the middle of the steel web panel, and is shown in Fig. 6. It is possible that lower JSDs would have occurred at the connection face because they typically decrease towards the regions away from the column center. However, the large diagonal cracks that developed in the connection suggest that the whole column width was mobilized, and thus JSDs measured on the web should be representative of the response at the connection face. As shown in Fig. 6, the steel web panel behaved in the elastic range up to 2.0% drift. When the peak load was attained at 3.0% drift, the measured JSD was 0.018 and 0.015 rad for the positive and negative loading directions, respectively. This level of shear deformation translated into significant damage in the joint region, characterized by a considerable number of diagonal cracks with widths exceeding 0.08 in., as shown in Fig. 4a. At 5.0% drift, the JSD slightly exceeded 0.02 rad, accompanied by severe joint damage (Fig. 4c).

Bearing distortions and column rotations adjacent to the hybrid joint were measured through linear potentiometers located within a distance of 14 in. from the steel beam flange, as shown in Fig. 7. The lateral load versus bearing distortion + column rotation response is shown in Fig. 8. Up to the peak load (3.0% drift), a stable response was observed with rotations below 0.02 rad. Beyond this point, bearing distortions + column rotations increased significantly due to slippage of the column reinforcing bars, leading to a peak rotation of 0.05 rad at 5.0% drift.

At peak lateral load, bearing distortions + column rotations represented approximately 50% of the total story drift, and during the last two loading cycles, these deformations accounted for almost the totality of the applied drift. Data gathered from strain gages confirmed that the steel beam remained in the elastic range throughout the test.

Stiffness Retention Capacity

The stiffness retention capacity of the subassembly was evaluated by normalizing the peak to peak stiffness for the first cycle at each drift level to that measured during the first cycle to 0.5% drift. A plot of the normalized stiffness versus drift is shown in Fig. 9. At first cracking, a 20% reduction in stiffness was observed. For the cycles between 1.0% and 2.5% drift, during which the specimen behaved either in the cracked elastic range or with only limited yielding, a gradual decrease in stiffness was measured. Beyond 2.5% drift, significant inelastic activity in the steel web panel and slippage of column longitudinal bars occurred, leading to a higher rate of stiffness degradation. At 5.0% drift, the specimen retained only about 20% of its stiffness at 0.5% drift. With regard to stiffness retention capacity for repeated cycles at the same drift level, the specimen retained at least 90% of its stiffness up to 3.0% drift. For the repeated cycle at 4.0% drift, a 20% loss of stiffness was observed due to large concentrated rotations at the beam-column interface.

Energy Dissipation Capacity

The energy dissipation capacity of the test specimen was evaluated by comparing the energy dissipated during each displacement cycle (EDi) to the energy dissipated by an equivalent perfectly elastic-plastic system (E_{EPi}) with stiffness equal to the initial stiffness of the specimen and strength equal to that corresponding to each drift level (Fig. 10).

The values for energy ratios at various drift levels are shown in Fig. 11. As can be observed, the energy ratio remained fairly constant up to 4.0% drift. Despite the visible inelastic deformations observed after 2.0% drift, there was no significant change in the energy dissipation ratio. Increase of pinching in the load versus displacement loops, primarily attributed to bearing distortions due to column bar slippage, was the main reason for this phenomenon. The energy dissipated during the repeated cycles at the same drift level was lower than that for the first cycle at each drift level.
Fig. 6 - Shear Distortion in Steel Web Panel

Fig. 7 - Location of Linear Potentiometers to Measure Bearing Distortion and Column Rotation

Fig. 8 - Lateral Load vs. Bearing Distortion + Column Rotation
Fig. 9 - Stiffness Degradation

Energy ratio = \( \frac{E_{Di}}{E_{Epi}} \)

Fig. 10 - Definition of Energy Dissipation Ratio

Fig. 11 - Story Drift vs. Energy Dissipation Ratio
FUTURE RESEARCH

Based on the severe column reinforcement slip that led to excessive bearing distortions during the first test, it is clear that new joint details are needed to increase joint bearing strength and control bearing distortions in RCS roof T-connections. For this purpose, the joint details in the second specimen will include vertical U-shaped stirrups, as used in the first specimen, and two small steel C sections running transversely above the top beam flange. Small holes, with a slightly larger diameter than the column bars, will be drilled in the web of the C section to allow the column longitudinal bars to pass through. The head of the bars will have a larger diameter than the hole and therefore, it will directly bear on the steel C section. A plan view of the joint details for the second test specimen is shown in Fig. 12.

CONCLUSIONS

This paper reports preliminary results from the test of one RCS roof beam-column T-connection subassembly under reversed cyclic loading. Three simple joint details were used and consisted of face bearing plates (FBPs), steel band plates (SBPs) and vertical U-shaped stirrups. The specimen showed a stable load versus displacement response up to the first cycle to 4.0% drift. The subassembly exhibited a connection bearing failure characterized by slippage of the longitudinal column bars through the connection which led to severe pinching in the load versus displacement hysteresis loops during the 4.0% and 5.0% drift cycles. For the next test specimen, the use of new connection details that include steel C sections is expected to improve connection bearing behavior.

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