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Ductility and Fracture of Joints with Panel Zone Deformation

Ductilité et mode de rupture d'assemblages avec panneau de renfort d'âme

Verformbarkeit und Bruch von Rahmenknoten mit Stegverformung

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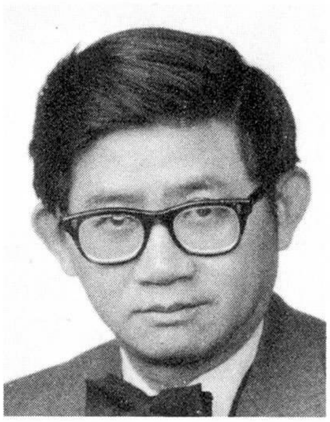

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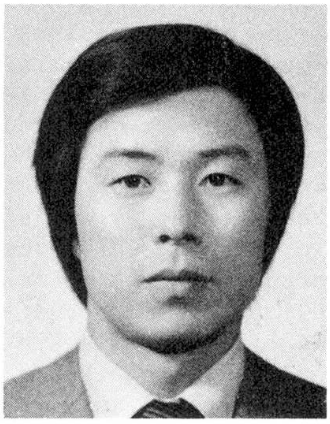

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SUMMARY
An experimental study of the inelastic behavior of beam-to-column joints with panel zone deformation has been carried out and selected results are presented. The factors examined include shear reinforcement of column web, horizontal stiffeners, and composite beam action.

RÉSUMÉ
Une étude expérimentale du comportement inélastique d'assemblages poutres-colonnes avec renforts d'âme a été menée, et les résultats intéressants en sont présentés. Les paramètres examinés sont le renforcement de l'âme de la colonne, les raidisseurs horizontaux et l'effet mixte de la poutre.

ZUSAMMENFASSUNG
Es werden ausgewählte Ergebnisse einer experimentellen Untersuchung über das unelastische Verhalten von Rahmenknoten mit Stegschubverformungen dargestellt (mehrstöckige Rahmen). Der Einfluss der folgenden Parameter wurde untersucht: Schubverstärkung des Stützenstegs, horizontale Aussteifungen und Verbundwirkung zwischen Stahlträger und Betondecke.

1. INTRODUCTION

Building structures are usually designed to satisfy both the serviceability and the strength requirements, a majority of which are specified in applicable codes. If a building is to be built in a seismic region, the overriding design concern is the effect of earthquake. The design practice in the U.S. requires that attention be given to such problems as (1) story drift at the code level earthquake forces, (2) stresses in members under working gravity load and code level earthquake forces (must be less than the code allowable stresses), and (3) response of the structure during a severe earthquake. The last problem requires a careful consideration of ductility and energy absorption capacity of the critical structural elements and of the overall structure.

A structural system that has been widely used in building construction and has performed reasonably well in laboratory testing and during actual earthquakes is the moment-resistant steel frame. The system has good energy absorption capacity, but its stiffness against drift is not high. In designing a moment-resistant frame, it is often necessary to use girders that are considerably larger than those required to satisfy the allowable stress criteria in order to control drift. At the code seismic force level, the stresses in these girders can therefore be substantially less than the allowable values. However, when such a frame is subjected to a major earthquake and is assumed to remain elastic, the lateral forces generated could be several times greater than the code forces. Inelastic action must therefore take place in the highly stressed regions of the structure. One such region is at the ends of the beams, where plastic hinges may form if the weak-beam, strong-column concept is followed in the design and if the joints are capable of transmitting the full plastic moment of the beams.* To satisfy the latter condition, the panel zone of the joint is often strengthened with shear reinforcement such as doubler plates. This increases, sometimes substantially, the fabrication cost. Some structural engineers therefore ask the question: If the girder is sized to meet a drift limitation, is it necessary to design the joint and the connection to develop the full plastic moment of the beam? The Uniform Building Code [1] gives the following guidelines:

> Connections: Each beam or girder moment connection to a column shall be capable of developing in the beam the full plastic capacity of the beam or girder.
>
> Exception: The connection need not develop the full plastic capacity of the beam or girder if it can be shown that adequate ductile joint displacement capacity is provided with a lesser connection.

The above "exception" implies that it is permissible to utilize the inelastic action of the panel zone of the joint to dissipate part of the energy input during an earthquake. The amount of inelastic deformation required of the joints is related to the characteristics of the earthquake ground motion and the properties of the frame. A complete inelastic seismic response analysis is necessary in order to determine the inelastic joint deformation and to evaluate overall performance of the structure. However, before such an analysis can be performed, the behavior of joints with panel zone deformation must be well understood and is properly represented by analytical models.

Among the various factors that affect the behavior of the panel zone, the following are considered to be significant: (1) the amount of shear reinforcement, (2) the presence or absence of horizontal stiffeners (or continuity plates),

*In this paper, a joint is defined as the entire assemblage at the intersection of the members, and a connection is only those elements that connect the member to the joint.

and (3) the details employeed in welding the shear reinforcement and stiffeners. Another problem that has received considerable recent attention is the effect of composite action of girders on joint and panel zone behavior. This is a complex problem, especially when the joint is subjected simultaneously to both positive and negative bending moments.

These problems have been studied in an experimental investigation carried out recently at the Fritz Engineering Laboratory of Lehigh University. The emphasis of the investigation is on the inelastic deformation capacity of the panel zone and the failure mode of the joint under cyclic loading.

2. DESCRIPTION OF TEST SPECIMENS

Three series of girder-to-column joints have been tested. The first series included four full-scale interior joints, three having shear reinforcement in the form of doubler plate and one reinforced. For the three specimens with shear reinforcement, the details of welding the doubler plate to the column varied. The second series, also included four interior joint specimens, examined the effect of horizontal stiffeners on panel zone deformation. The third series studied the behavior of both interior and exterior joints with composite girders. In this series three full-scale specimens, all without shear stiffening, were subjected to cycles of repeated and reversed loading until failure. In this paper, the results of four selected test specimens, two from the first series and one each from the second and third series, are presented and compared with reference to the effects of (1) shear reinforcement, (2) horizontal stiffener, and (3) composite girder action.

All the test joints were made of A36 steel with a nominal yield stress of 250 MPa. The girder flanges were fully welded to the column and the web was bolted to a connection plate with ASTM A325 bolts. The girders were sized to provide sufficient flexural and shear strength to force severe yielding to occur in the panel zone and its boundary elements when no shear reinforcement was added. The web connection was designed to carry all the vertical shear. The three bare steel specimens which were designated as Joints A, B and C, had the same general dimensions and member sizes, as shown in Fig. 1. The composite joint was designated as Joint D, the details of which are given in Fig. 2.

2.1 Joint A

This was the only joint that was reinforced by both doubler plate and continuity plates. The doubler plate was 12.7 mm (1/2 inch) thick and had a nominal yield stress of 345 MPa. It was welded to the column by fillet welds. This plate together with the web of the column was sufficient to resist the shear transmitted to the joint when plastic hinges formed in

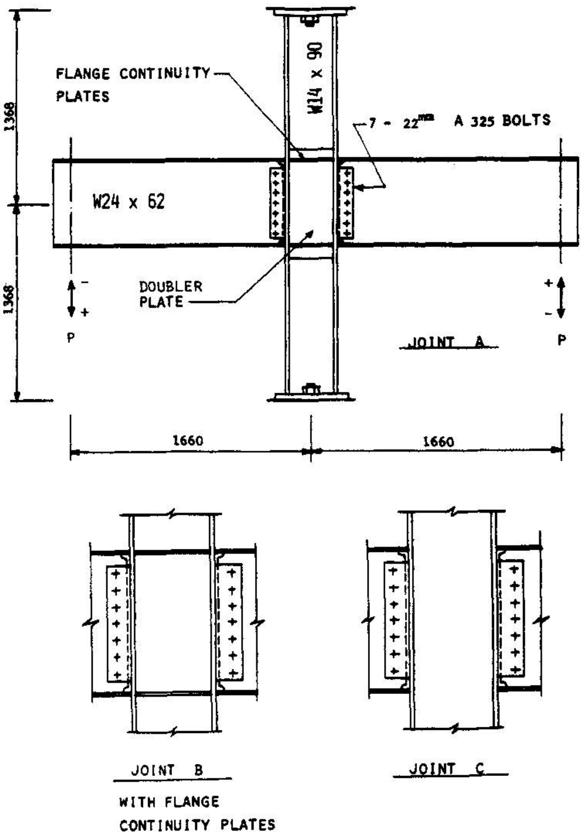

Fig. 1 Dimensions and Details of Joints A, B and C

both girders. The calculation was based on a shear yield stress of $0.68\sigma_y$ not the von Mises yield stress of $0.58\sigma_y$. (See Ref. 2 for an explanation of the selection of the yield stress.)

2.2 Joint B

This joint was identical to Joint A except that no doubler plate was provided. The joint ductility was expected to be due largely to shear yielding of the panel zone.

2.3 Joint C

Neither doubler plate nor continuity plates were provided in this joint. The results of this test can be compared directly with those of Joint B to evaluate the effect of continuity plates.

2.4 Joint D

This specimen represented an interior joint of a six-story, two-bay prototype test building. The composite slab was cast on a metal deck which was connected to the girder by headed shear studs. The concrete was lightweight with a 28-day compressive strength of about 34 MPa (5000 psi). Although the member sizes of this specimen were not the same as those of the other joints, a qualitative study of the effect can be made in terms of strength and panel zone deformation capacity.

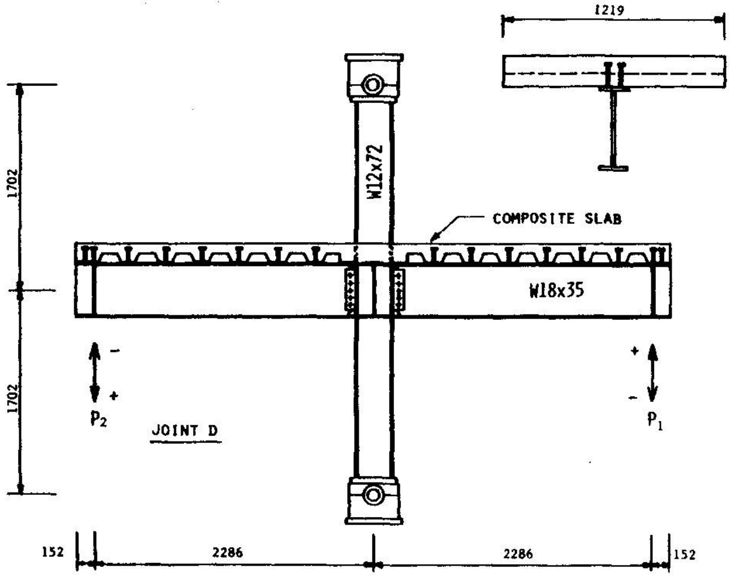

Fig. 2 Dimensions and Details of Joint D

3. EXPERIMENTAL BEHAVIOR AND RESULTS

3.1 Test Procedure

The specimens were tested by repeatedly applying loads in opposite directions to the beams. The direction of each load was also reversed. For Joints A, B and C, the testing was controlled by panel zone shear deformation, except at the early stage when load control was used. The panel zone deformation was measured either by a diagonal gage or by rotation gages attached to the column web. For Joint D, the vertical deflections at the load points were used as the control, and the deflections of the four corners of the panel zone were measured independently. The measured deflections were then converted to panel zone rotation.

3.2 Joint A

In testing the specimen, load increments of 45 kN per beam were used until the panel zone deformation reached approximately 1.0%. The remainder of each cycle was achieved by loading until the diagonal cycle gage indicated increments of approximately 0.5% additional rotation. The loading was continued up to a maximum panel zone rotation of 2.7%, at which very extensive yielding was observed in the two beams just outside of the joint. It appeared that any other loading of the beams beyond this level would produce only limited additional panel zone deformation. A visual inspection of the specimen after seven load cycles showed small cracks forming in the beam flange connection welds. The test was stopped after seven cycles.

The maximum load reached during the final cycle was 495 kN, which was very close to the plastic limit load of the beam, 488 kN. The hysteresis loops of the first, second, third and seventh cycles are shown in Fig. 3. They exhibit the usual stable characteristics associated with steel structures prior to failure due to fracture or instability. There was very substantial strain hardening which occurred almost as soon as the critical region of the panel was yielded.

3.3 Joint B

The specimen was tested with the same load and panel zone deformation increments as Joint A. The removal of the doubler plate reduced greatly the shear resistance of the panel zone and the maximum beam load. Most of the yielding therefore occurred in the panel zone. In fact, the purpose of this test was to demonstrate that the panel zone had adequate ductility and could be subjected to large cyclic distortions without failure.

A total of seven inelastic load cycles were applied, and the range of panel zone rotation was between +4% and -6.2%, the latter was limited by the stroke of the jacks used to load the beams. There was no visible distress in the beam flange welds at these large distortions. The results of the first three cycles as well as the last cycle are shown in Fig. 4. Strain hardening of the panel zone was also very pronounced and the test loads were found to be substantially higher than that calculated by the von Mises criterion.

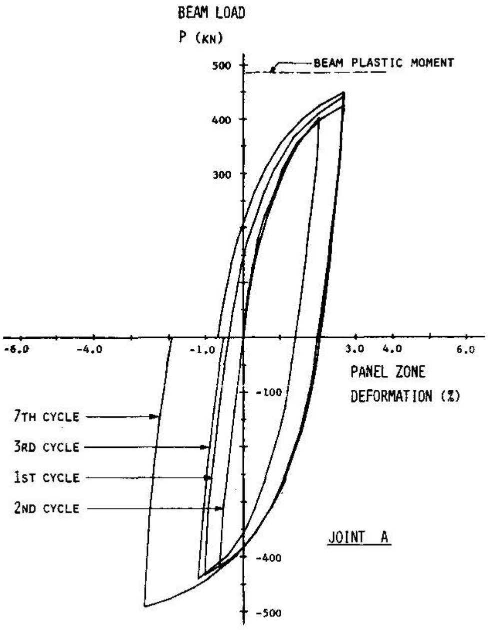

Fig. 3 Load-Deformation Curves of Joint A

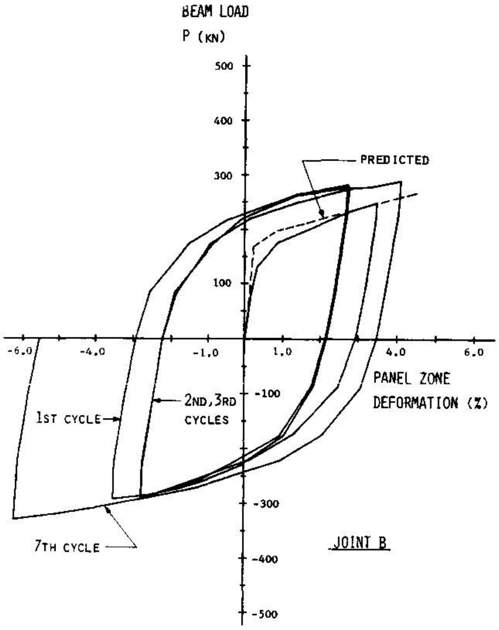

Fig. 4 Load-Deformation Curves of Joint B

3.4 Joint C

The same procedure was again followed in this test. Because earlier studies on joints without continuity plates had indicated significantly less ductility, it was decided for this test to reduce the range of panel zone rotation to about 3.0%. In the first and second load cycles, this joint behaved very much like Joint B, but the removal of the continuity plates apparently had some effects on stiffness. The specimen exhibited a well-defined panel zone for resisting shear. This is illustrated in Fig. 5, which also shows the yield lines in the column flanges opposite to the beam flange welds. The specimen failed at the fourth cycle by a crack through one of the column flanges at the edge of a beam weld.

The results of the test are given in Fig. 6. The decreased slope of the load-deformation curve before fracture indicates that cracks may have developed in the column flange during the previous cycle.

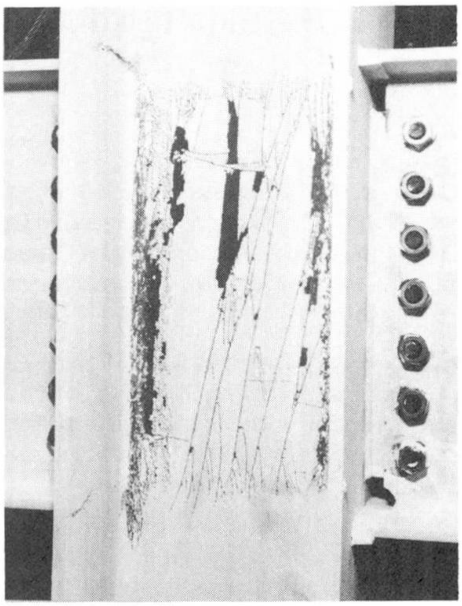

Fig. 5 Panel Zone Yielding of Joint C

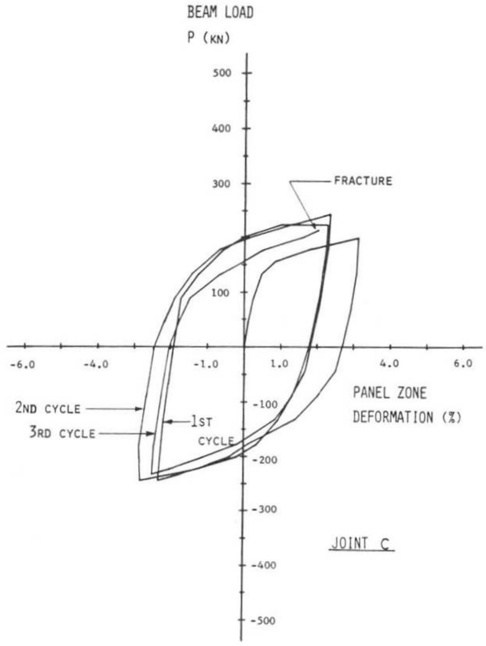

Fig. 6 Load-Deformation Curves of Joint C

3.5 Joint D

A total of 37 load cycles, 24 of which caused inelastic deformation of the panel zone, were applied to the joint. The cycles involved continuously increasing deflections of the load points on the beams, which were used to control the test. The concrete slab cracked in tension very early but continued to provide compressive resistance when the direction of the beam moment was reversed. The specimen failed when cracks developed near the coped holes in the tension flanges of the beams. Such a crack is shown in Fig. 7.

This joint is similar to Joint B in that the panel zone alone was insufficient to resist the shear. Substantial inelastic deformation must occur in the panel zone. In Fig. 8 the total beam load ($P_1 + P_2$) is plotted against the panel

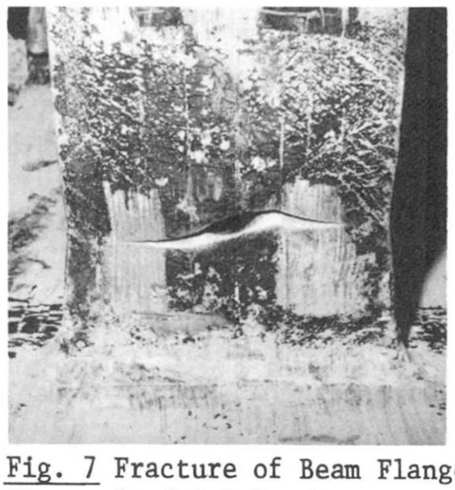

Fig. 7 Fracture of Beam Flange in Joint D

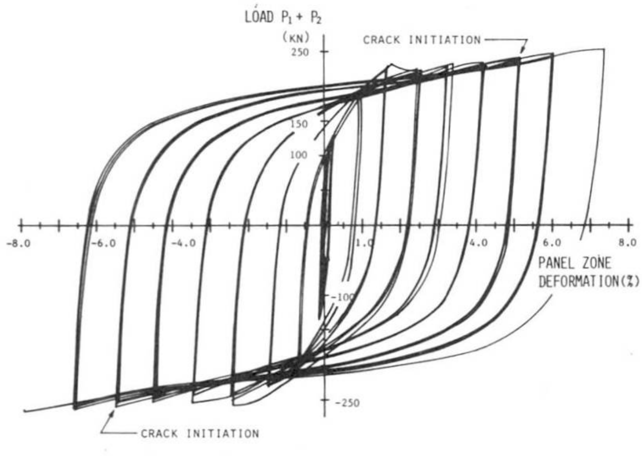

Fig. 8 Load-Deformation Curves of Joint D

zone rotation for all the load cycles. Crack initiation in the beam flange was observed at a panel zone rotation of about 5%, and the maximum rotation achieved was more than 6%.

4. DISCUSSION

Joint A represents the situation in which the designer wishes to utilize both the panel zone rotation and beam yielding for energy absorption. This concept has the advantage of reducing the ductility demand on the beam and its connection to the column flange, thus producing a more balanced design. The panel zone rotation achieved in the test was 2.7%. Based on this value and the theoretical calculations of the inelastic deformation capacity of the beams, a story drift of more than 4.5% has been estimated.

The W24 x 62 beam is unique in that a substantial portion of its plastic moment is contributed by the web. Based on the measured yield stresses of the flange and web of the beams of Joint A, this contribution is found to be 40%. A generally accepted concept of designing connections with fully welded flanges and bolted web is to assume that all the bending moment is resisted by the beam flanges and all the shear resisted by the web. To satisfy this condition, the beam flanges must strain harden sufficiently to make up the difference between the full plastic moment of the section and the plastic moment provided by the flanges. This may become a severe problem for sections with a large portion of the plastic moment provided by the web. However, the test results of Joint A do not seem to indicate this to be particularly serious.

Another feature of Joint A is the use of fillet welds in welding the doubler plate to the column. This procedure, which is less costly, appears to be a satisfactory alternative to full penetration welding.

Joints B and C, both without shear reinforcement, simulate the joints in a frame in which the beams are over-sized for drift control and inelastic action of the panel zone is expected to absorb the energy input. The highly ductile behavior of the panel zone in Joint B indicates the possibility of utilizing shear yielding for energy absorption. The behavior of joints with panel zone yielding can be predicted by the method proposed by H. Krawinkler [3]. In this method, the inelastic deformation of the panel zone is assumed to occur in three stages: shear yielding of the web panel, formation of plastic hinges in the column flanges, and strain hardening of the web panel. This method has been applied to predict the load-deformation relationship for Joint B and the results are shown in Fig. 4. The web panel is fully yielded at a load of 171 kN, but, because of column flange yielding and strain hardening, the maximum load reached in the test was 325 kN, an increase of 90%.

The relatively poor performance of Joint C is a problem of concern and is being carefully examined. A finite element study made on joint geometry has revealed that there is a severe stress concentration in the column flange where the beam flange is attached in the region adjacent to the web when there are no continuity plates. It appears that adequate ductility is very much dependent on having continuity plates of some size in the panel zone.

The results of Joint D test again shown highly ductile behavior of the panel zone. Very substantial strain hardening also occurred, which allow the adjoining beams to yield extensively before fracture of the tension flanges. The envelope or skeleton curves of the hysteresis loops of Fig. 8 are shown in Fig. 9, where the theoretical prediction based on Krawinkler's method is also given. The composite action of the slab makes it difficult to define a proper panel zone height. The results given in Figs. 8 and 9 assume a panel zone height equal to the distance between the continuity plates. The actual height may be larger. The theoretical prediction, which neglects the contribution of the composite slab, is shown to be very conservative.

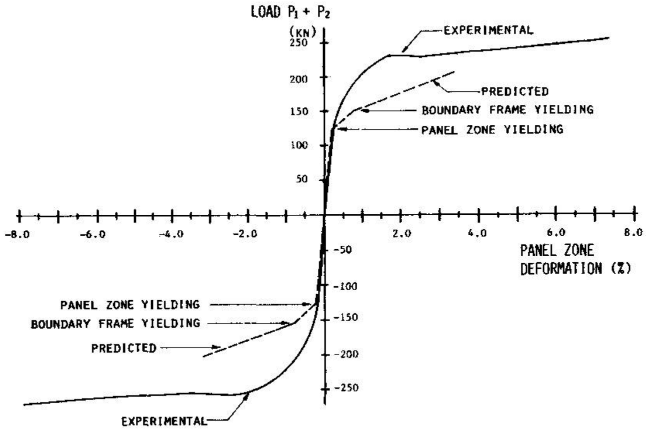

Fig. 9 Skeleton Curves of Joint D

5. CONCLUSIONS

The following conclusions may be drawn from the results presented; they are applicable to joints with dimensions and member sizes comparable to those of the test specimens.

1. The web panel and its boundary elements in a joint with continuity plates can deform inelastically through large shear distortions. A panel zone rotation of 5 to 6% may be achieved with substantial strain hardening.
2. The ductility of joints can be severely imparied when continuity plates are not provided. The joint may fail by cracks through the column flanges adjacent to the beam flange connection welds.
3. For joints designed to develop the plastic moment capacity of the beams, it may be beneficial to allow limited yielding in the panel zone in order to reduce the ductility demand on the beams and the connecting elements.
4. When over-sized beams are used for drift control, shear reinforcement of the column web may not be necessary if sufficient panel zone ductility is available.
5. The panel zone in a composite beam-to-column joint can also behave ductilely and it is possible to achieve an inelastic rotation comparable to that of a non-composite joint.

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