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SEISMIC MOMENT CONNECTIONS FOR MOMENT-RESISTING STEEL FRAMES

by

EGOR P. POPOV

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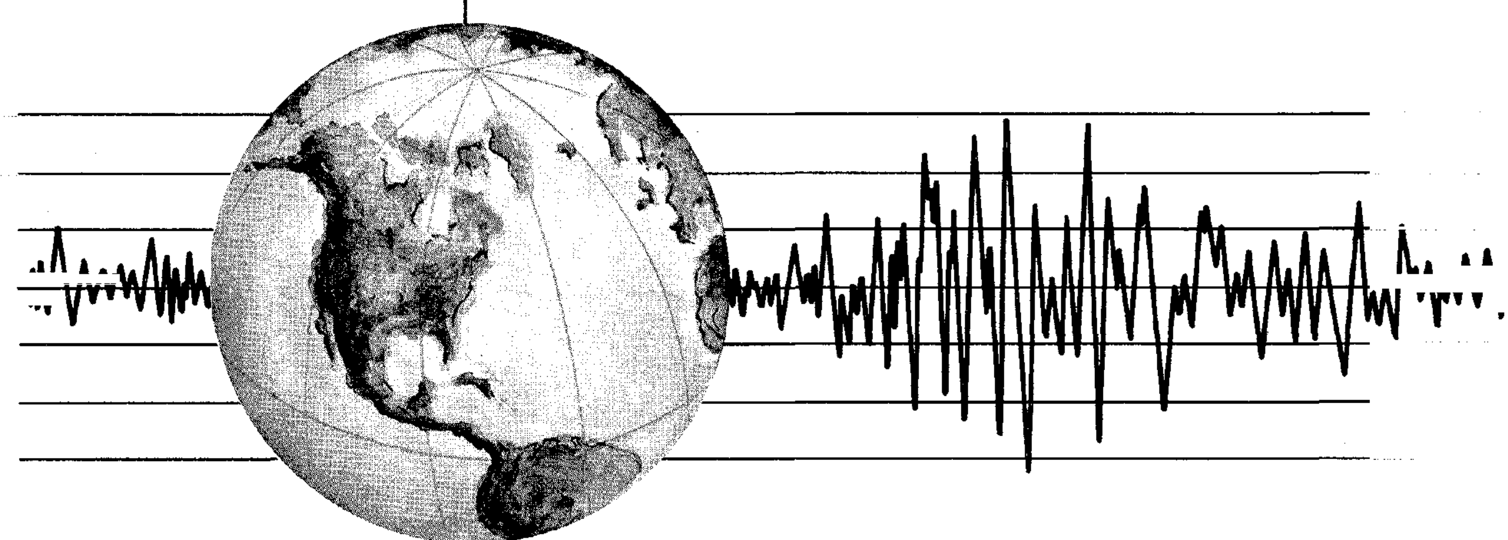

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SEISMIC MOMENT CONNECTIONS
FOR MOMENT-RESISTING STEEL FRAMES

by

Egor P. Popov
Professor of Civil Engineering

Report to

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and
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ABSTRACT

This report is prepared to be Chapter 6 of the upcoming ASCE Manual on Beam-to-Column Building Connections currently under review by members of the Monograph Task Committee of the Committee on Structural Connections of the ASCE Structural Division.

This chapter provides an overview of the state of the art for the design of steel moment connections for regions of high seismic risk. The need for designing such connections to be ductile with the capacity to sustain full load reversals is indicated first. The generally accepted approach of "strong columns-weak girders," i.e., designing the joints to develop inelastic activity in the connections and beams rather than columns, is adhered to throughout the chapter. A major section of the report is devoted to presentation of experimental results to illustrate the observed behavior of beam-to-column connections and column panel zones under severe cyclic loadings simulating extreme seismic conditions. Procedures are given for seismic moment joint calculations pertaining to flange beam connections, web connections, and panel zone design. Typical seismic moment connections of beam-to-column flanges as well as to webs, and column splices are illustrated. The chapter concludes with an indication of some of the problems requiring further research.

TABLE OF CONTENTS

List of Figures

CHAPTER 6 - SEISMIC MOMENT CONNECTIONS

By Egor P. Popov
Department of Civil Engineering
University of California
Berkeley, CA 94720

6.1 INTRODUCTION

6.1.1 Design Philosophy

Building frames for static as well as wind and earthquake loads are currently being almost exclusively analyzed on the basis of elastic concepts. The elastic approach dominates the current AISC Specifications [1980] for structural steel design, and is followed in the Uniform Building Code [UBC, 1982], which is widely used for seismic resistant design of steel structures. A gradual transition to plastic methods of analysis and to the load and resistance factor design (LRFD) [Galambos, 1976,1981] is likely to take place in the future.

However, in seismic design, unlike analysis, a modified approach for sizing connections and joints is usually employed that recognizes inelastic behavior. This approach, which will be described in this chapter, stems from the fact that field observations in the aftermath of severe seismic excitations indicate that structural deformations far beyond those calculated on the basis of elastic analysis are found to occur. This kind of behavior is accentuated further by the fact that the currently specified code provisions underestimate the force magnitudes likely to be generated by a seismic disturbance. During a major earthquake, it is considered feasible to tolerate some inelastic action causing permanent deformation in members. The permissible ductile deformation of a frame beyond its elastic limit is dependent upon absorbing and dissipating the energy input caused by the earthquake. The inelastic (hysteretic) behavior of the frame effectively dampens the motion and enables the structure to survive.

For the above reasons, members and joints must be designed and detailed to be capable of deforming well into the inelastic range without local failure or frame instability. Frame designs fulfilling such requirements are referred to as <u>ductile</u> moment-resisting frames; they are used in regions of high seismic risk, and their connection details are discussed in this chapter. For such frames it is recommended practice [SEAOC Recommended Lateral Force Requirements, 1980] to size the members so as to ensure that plastic hinges will form in the girders rather than the columns. This approach of "strong columns-weak girders" is virtually universally accepted by design engineers. Therefore, one of the main problems is associated with the design of connections and splice regions so that they are compatible with the formation of plastic hinges in the girders.

A number of steel framing systems can be used to develop the required lateral strength, stiffness, and ductility for resisting forces generated by earthquakes. In most of these systems connections between columns and beams must be moment-resisting. This type of a connection occurs in moment-resisting framing which is by far the most widely used structural steel framing system in seismic design.

An example of a typical ductile moment-resisting frame is shown in Fig. 6.1; a schematic diagram for this type of frame is given in Fig. 6.2. (The cross-hatched members in the diagram indicate possible structural subassemblages for experimental studies.) The lateral integrity of such frames depends entirely on having beam-column connections of sufficient strength as well as ductility for resisting seismic forces.

In the literature on seismic connection design, a specialized terminology has evolved that will be adhered to as much as is practical in this chapter. In a few instances it differs from that used in conventional design. The relevant terms have the following meaning:

Joint is the entire assemblage at the intersections of the members.

Connection consists only of welds or bolts interconnecting the elements of a joint.

Shear tab or web plate is a plate welded directly to the column for making a bolted or welded shear connection to the beam.

Stiffener or flange continuity plate is a plate, usually placed in a horizontal position, welded to the inside faces of the column flanges as well as to the column web.

Connecting plate is a plate placed similarly to that of a stiffener plate, but not welded to the column web. Such plates usually cannot be used in resisting seismic loads.

Doubler plate is a plate placed parallel to the column web occupying the space between the column flanges and the stiffener plates. In some designs such a plate extends at both ends beyond the stiffeners above and below beam flanges.

Panel zone is the part of a column web forming the joint, and includes the doubler plate if there is one.

6.1.2 Moment Joint Design Problem

The basic problems encountered in a moment-resisting joint of a moment-resisting frame may be noted from the diagram in Fig. 6.3. The connections of girders to the column flanges must transmit bending moments and shears, which may act in either direction. In many instances the bending moments in the beams caused by seismic loading exceed those caused by gravity loads, which results in beam bending moments having the same sense on both sides of a column (shown by matching dark or open arrows in the figure). During a seismic event, a flange force in a beam on one side may pull the column, whereas the corresponding flange force on the other side would push the column, or vice versa. A part of these flange forces are transmitted directly to the column web, while the larger remaining part is transferred to the column web stiffeners, which in turn must be attached to the column web. The transfer of flange forces to the column web subjects the column panel zone to large shearing forces. Due to the presence of gravity forces, the beam shear forces tend to remain acting in the downward direction. Similar problems are encountered in moment connections to box columns and column webs. These problems will be commented upon in Sections 6.4.1 and 6.4.2, respectively.

As a building frame experiences dynamic reversing motions due to an earthquake, as shown diagramatically in Fig. 6.4, joints may become subjected to an appreciable number (5 to 10) of severe cyclic reversals. From the point of view of statics, these deformed states correspond to collapse mechanisms. However, under dynamic conditions, it is more appropriate to refer to these deformed states as mechanism motions; these motions dissipate energy in the plastic regions of the members and dampen the vibration of a frame. Studies of this problem [ATC Publication ATC 3-06, 1978] indicate that for severe exposure the anticipated story drifts, including inelastic deformations, are on the order of 1.5 to 2 percent. Such a criterion, along with the kinematics of a frame collapse mechanism, provides guidance for the required rotations at the joints. Therein is the essential difference in the behavior of joints in static compared with seismic loadings: joints in seismic design must be capable of resisting forces in either direction. Therefore, for example, the column stiffeners must have capacity for resisting either tensile or compressive forces.

In designing seismically resistant joints it is also very important to recognize that they must be ductile. This requirement can be arrived at from studying the diagram in Fig. 6.5, which shows response spectra for a selected massive earthquake of structures with different natural periods of vibration and amounts of available ductility. If the structures are elastic, the upper curve is applicable. Therefore, for example, if the natural period of a particular structure were 2 seconds, the base shear coefficient, which through its mass directly relates to the lateral force, would be about 0.35. On the other hand, the coefficient prescribed by the UBC [1982], defined in the figure by the dashed line at approximately threshold of yielding, is only 0.066. This huge discrepancy between the code prescribed lateral force and the one that would develop in an elastic structure can be reconciled only if a structure is ductile.

Thus, for the earthquake of large intensity shown in Fig. 6.5, if a structure could sustain without a loss in strength six times the story deflections that would occur at yield (i.e., if $\mu_\delta = 6$), the structure would survive. This extreme frame ductility requirement would be smaller if an occupancy importance factor on the order of 1.25 or 1.5 were applicable to the structure, which would move the dashed curve upwards in this ratio. Nevertheless, it is important to emphasize that the appraisal of connections and joints for seismic design must include not only their strength, but also their ductility characteristics. In the next section, the usual experimental methodology for determining these characteristics is described.

6.1.3 Test Procedures for Joint Evaluation

Experimental work for evaluating the behavior of connections and panel zones in moment-resisting joints has usually been done by applying to the joints progressively increasing cyclically reversing loadings. As the rate at which such loadings occur during earthquakes on individual members is relatively slow, it is possible to perform experiments in a quasi-static manner, i.e., by slowly imposing cyclically reversing forces or displacements. A typical example of a test specimen and an applied sequence of cyclic end displacements for determining connection behavior may be seen in Fig. 6.6. In an actual experiment, the column stub is bolted to a steel anchor box, and a double acting actuator applies either the prescribed cyclic force, $\pm P$, or prescribed displacements, $\pm\Delta$. Usually, the specimen is initially subjected to prescribed slowly applied cyclic forces, but as plastic behavior at the beam-column interface develops, the experiment is controlled by applied displacements.

Experiments on specimens such as shown in Fig. 6.6a provide information on the cyclic behavior of the connection between the beam and the column as well as on the behavior of the beam in the critical region.

The simplest experimental setup for studying the behavior of a panel zone is shown in Fig. 6.7. In the presence of an axial force P, the cyclic shears V_1's are applied acting simultaneously in opposite directions and impose a large shear on the panel zone. This is a statically determinate setup, so that the evaluation of the test results is direct. In the more comprehensive experiments, frame subassemblages such as shown cross-hatched in Fig. 6.2 are used. Depending on the support conditions (see Fig. 6.14), the subassemblages for the interior joints can be statically indeterminate. For either type of specimen, however, the applied force or forces are cycled in a manner analogous to that shown in Fig. 6.6b. In some experiments, a large displacement(s) is applied initially. Small-scale experiments on shaking tables are used for corroborative evidence only.

6.2 EXPERIMENTAL OBSERVATIONS ON JOINT BEHAVIOR

6.2.1 Cyclic Behavior of Beam-to-Column Connections

The cyclic behavior of moment-resisting connections under severe loading conditions appears to have been first investigated on small specimens by Popov and Pinkney [1967, 1968a, 1968b, 1969]. More comprehensive studies of their behavior as parts of one-third scale interior column subassemblages (see Fig. 6.2) were reported by Krawinkler *et al.* [1961], and on reduced scale one-bay frames by Carpenter and Lu [1973]. Subsequently, the behavior of moment-resisting frames with realistic joints on reduced scale models was investigated on a shaking table by Clough and Tang [1975]. However, it is difficult to simulate at small scale the connection details used in practice consisting of large welds and bolts. For this reason, especially in this section dealing with connections themselves, the results for connections-to-column flanges are drawn principally from full-size experiments of Popov and Stephen [1970,1972].

In order to determine the behavior of full-size moment connections to column flanges under severe cyclically reversing loading, eight specimens of the type shown in Fig. 6.6a were fabricated and tested in the manner indicated in Fig. 6.6b. The beams used in these experiments were either W18 × 50 or W24 × 76 sections. Two types of connection details were investigated, as shown in Fig. 6.8. In the welded connection, Fig. 6.8a, the two bolts simulated the erection bolts commonly used in construction. In the connection shown in Fig. 6.8b, the beam web was attached to the shear tab with high-strength bolts. In both details the flanges were welded to the columns with full penetration welds. The beams were coped at the top for back-up plates, and at the bottom to allow a continuous flange weld. All the specimens tested were fabricated in a production shop. Nominally the steel was A36 material, although the W18 × 50 beam flanges had a yield of 48 ksi. A325 bolts with washers were used throughout. One of the beams had no web connection whatsoever, and was attached to the column stub only by means of full-penetration welds of the flanges. This experiment was useful in showing that a considerable amount of shear can be transmitted through the flanges.

Representative results for two W24 × 76 specimens are shown in Fig. 6.9. Measured by the attained deflections prior to any loss in strength, good ductility of the specimens is observed in both cases. Due to strain hardening of the steel, the strengths exceeded the plastic strengths determined from ideal plastic theory. It is noted, however, that the welded specimen, Fig. 6.9a (see detail in Fig. 6.8a), exhibited superior ductility. The tip deflection reached approximately 6 in. (150 mm) without a loss in strength. The ductility of the specimen with a bolted web (Fig. 6.8b) shown in Fig. 6.9b, while perhaps adequate for most applications, is inferior to the one with a welded connection. However, some of the specimens with bolted webs (not shown) were nearly as good as those of the welded type. For this reason the less expensive connections with bolted webs are favored in practice. For the rotation ductilities involved in these experiments, flange buckling was not a serious problem. A correct welding and bolting sequencing is very important in all cases to minimize residual stresses.

The appearance of a welded connection at the completion of a test may be seen from the photograph in Fig. 6.10. The full participation of both the flanges and the web should be noted by observing the scaled off whitewash. The corresponding appearance of a specimen with a bolted web may be seen in Fig. 6.11. In comparison with the welded connection, it can be seen that the beam web was not fully engaged in resisting the applied cyclic loads. This is due to the fact that at large cyclic loads some slippage between the shear tab and the web often develops. At the end of several cycles on this type of connection, explosive flange failures by tearing out from the column stub flanges were observed. A photograph of this type of failure for such a case is shown in Fig. 6.12. However, these failures occurred only after a number of large cyclic load reversals similar to those shown in Fig. 6.9b were applied to the specimen.

Connections of beams to box columns are made in the same manner as described above. Moment connections of beams-to-column webs are similar, except that appropriate stiffener (flange continuity) plates and shear tabs must be provided for attachment. This kind of a connection is discussed in Section 6.4.2.

In passing, it may be of interest to note from Fig. 6.13 the type of failure that can take place in a welded connection with flange plates. However, in the illustrated case, the fracture occurred after 18 severe cycles, and whether this extent of low cycle fatigue is important in seismic design is debatable. This type of flange connection is now seldom used in practice. More frequently, shop-welded flange plates are connected to the beam flanges by means of high strength bolts. Some comments on this kind of a connection are made in Sections 6.4.1 and 6.4.2.

6.2.2 Cyclic Behavior of Panel Zones

In the design of buildings for seismic exposure, the Seismology Committee of the Structural Engineers Association of California in its Recommendations [1980] imposes a limitation of 0.005 times the height for story-to-story drift due to design seismic loads. More stringent recommendations are made by the Applied Technology Council [1978], which requires some structures to sustain up to four times the above story drifts without loss in strength. The three generally recognized factors which contribute to the drift are the joint rotation due to flexure of the beams, bending of columns, and axial deformation of the lateral resisting elements. The shear deformation of the column panel zone must be added to these factors. Attention to this problem in the inelastic range was first drawn by Krawinkler *et al.* [1971] and Bertero *et al.* [1972,1973] based on experimental and analytical studies of subassemblages. In these experiments, subassemblages to one-third scale of a 20-story moment-resisting frame were investigated. The selected models for an interior column subassemblage were of the type shown cross-hatched in Fig. 6.2. Two series of these experiments were specifically directed toward the investigation of the panel zone behavior. The third series of experiments on similar subassemblages emphasized the hysteretic behavior of plastic hinges in columns.

In the first series of experiments [Krawinkler *et al.*, 1971], one of the subassemblages approximately modeled the conditions existing at the fifth floor; the other, at the 17th floor. The general features of the idealized model for an interior column subassemblage are shown in Fig. 6.14. For both models, ℓ's were 160 in. (4.06 m), and h's were 80 in. (2.03 m). The column for the upper floor model was W8 × 24, and for the lower, W8 × 67. The corresponding beams were B10 × 15s for the lighter model, and B14 × 22s for the heavier model.

Each model was tested by first applying an appropriate axial force P to the column and simulated gravity forces G_1 and G_2 to the beams. Thereafter, two of the models (one of each type) were tested with a progressively increasing cyclically reversing horizontal force H or displacements in a manner analogous to that shown in Fig. 6.6b. The other two models were tested by initially applying a large displacement followed by a cyclic sequence of load applications.

During application of the above forces, the corresponding moment and shear diagrams for the column were as shown in Fig. 6.15. Since the moments from beams are transferred to the column mainly through beam flanges, the shear in the panel zone is large and nearly constant; this can induce significant shear deformations of the panel zone. Precisely this behavior was observed in the model for the 17th floor. The nature of the shear deformation of the panel zone in advanced stages of the experiment may be seen from Fig. 6.16.

Large panel deformations have an important consequence for the story drift. The effect of this deformation alone may be seen from Fig. 6.17. In extreme cases, panel zone deformations may contribute to the story drift as much as joint rotation, which is controlled by beam size. In the latter case, it sometimes becomes necessary to use larger beams than required for strength to reduce joint rotation. Therefore, reinforcement of the panel zone to reduce story drift becomes essential in some frames.

To gain some insight as to the extent of possible deformations of unreinforced panel zones, the hysteretic loops for two of the frames discussed above may be examined. These are shown in Figs. 6.18 and 6.19. In both these figures, the difference (unbalance) of beam moments at the connections is plotted against the average angle of shear distortion in the panel zone. The consequences of a ductile, but excessively large, panel deformation of a frame such as that shown in Fig. 6.18 are detrimental to frame strength. Note that in the thicker and larger panel zone of the frame with a heavier column, Fig. 6.19, the shear distortions are about ten times smaller than in the lighter column. In order to reduce panel zone deformation to a desirable level and thereby increase the strength of a frame, the panel zones must be reinforced with doubler plates. Without such reinforcement, the calculated strength (by conventional procedures) may overestimate the actual capacity of a frame. For seismic design, the required frame ductility must be principally developed in the beams.

In the second series of experiments [Bertero *et al.*, 1973], the problem of panel zone reinforcement was considered. Again, one-third scale subassemblages having the geometry shown in Fig. 6.14 were used, but the panel zones were reinforced in two of the specimens. For the more conventional case, a single doubler plate was employed. The column had a W8 × 24 section and, as before, the beams were B10 × 15s. For this specimen, a 1/4 in. (6 mm) doubler plate was fitted next to the column web between the stiffeners and the column flanges. The attachment of the doubler plate to these elements was made with 3/16 in. (5 mm) fillet welds. From the experiment on this specimen, it was determined that this plate with small fillet welds was fully effective in increasing the shearing strength and stiffness of the panel zone. In the other experiment, each of the two plates were placed symmetrically about 2 in. (50 mm) away from the column web. The distortion of these plates during the test was found to be significantly smaller than that of the column panel zone web. Therefore, these plates were less effective than the column web in inhibiting the panel zone deformation. Hence, a better design is obtained by placing the doubler plates close to the column webs.

In the related third series of experiments [Popov *et al.*, 1975], the same type of subassemblages that were used in the first two series of experiments were investigated. Four of the specimens had doubler plates next to the column web fitted between the stiffener plates and the column flanges. In all of these experiments the beams were W12 × 31s, and in three of the subassemblages the columns were W8 × 48s, and for one specimen the column was W8 × 28. For two of the W8 × 48 columns, 1/4 in. (6 mm) doubler plates on both sides of the column web were used; in the third W8 × 48 column, a single 7/8 in. (22 mm) doubler plate was provided; and for the W8 × 28 column, 3/8 in. (10 mm) doubler plates on both sides of the column web were employed.

These experiments were designed to determine the hysteretic behavior of plastic hinges in the columns, and both the beams and the panel zones were deliberately overdesigned. In all cases, the doubler plates were found to be effective in minimizing the distortion of the panel zones. Inelastic action developed, however, in all cases due to the fact that much larger column moments were induced than would be expected from simple plastic analysis, assuming ideal plastic behavior. Under cyclic loading, steel strain-hardens.

It must be emphasized that in the three series of experiments described above, the specimens were of modest size. Therefore, extrapolation to designs utilizing thick material and large welds must be done with great care. Experience has shown that in large, highly restrained joints, lamellar tearing might develop unless proper welding sequence and inspection procedures are strictly followed.

The question of the correct manner for calculating the required size of the doubler plates and their attachment to the column remains controversial, and is subject to re-examination. Some remarks on sizing the doubler plates will be given in Section 6.3.3. Comments on the recently completed tests by Slutter [1981] on full-size panel zone specimens follow.

These experiments were designed to demonstrate that fillet welds of the doubler plates and minimum size stiffeners are sufficient to develop the code capacity of the panel zone. In this program four specimens having the configuration shown in Fig. 6.7 were tested. It will be recalled that this is a statically determinate setup, requiring a minimum of data reduction for interpreting the results. For comparison, one of the specimens had no doubler plate; this specimen exhibited an early inelastic activity and did not attain the required strength. However, due to the strain-hardening of the steel, some increase in the joint capacity continued into the 2 percent range of panel zone deformation.

Secimen 3, which had the detail shown in Fig. 6.20, was perhaps the most successful. Its doubler plate of Grade 50 steel was thin, the welds small, and yet its hysteretic behavior, shown in Fig. 6.21, was very good. It is to be noted that in this specimen the doubler plate extended beyond the stiffener (flange continuity) plates. Doubler plates fitted between the column flanges and the stiffener plates are more common and usually larger welds than in the above tests are used around doubler plates. It should be carefully noted that these experiments were carried out without applying any axial column load. The behavior of joints with a more accurate simulation of the conditions which develop in a frame, i.e., the presence of both the axial force P and the plastic beam moments M_p's may lead to significantly different results. In the presence of a large axial force P, the panel zone may develop unacceptably large deformations. The deleterious effect of the axial column load on the behavior of plastic hinges in somewhat related experiments [Popov *et al.*, 1975] was found to be large.

Some additional experimental information on the behavior of doubler plates extending beyond the stiffeners is reported by Becker [1971]. No full-size experiments on the more conventionally fitted doubler plates between stiffeners appear to have been made.

In moment connections of beams to box columns or to the column webs, rather than flanges, the panel deformation problem does not usually arise. In either case, there are two column plates parallel to the beam web and these plates are usually thick, hence the induced shearing stresses are small.

No comprehensive research for determining stiffener plate size for specific application in seismic design appears to be available. Generally, the available recommendations for monotonically applied loading, such as those given in Chapter 4, are followed. Design should include the consideration of stiffener buckling, which usually is guarded against by assigning the conservative width-thickness ratios recommended for plastic design in Part 2 of the AISC Specifications [1980]. No specific criteria have been developed for web buckling in the panel zone. In most instances, however, the presence of other members framing into the panel zone prevent this from developing.

6.3 SEISMIC MOMENT JOINT CALCULATIONS

The design of moment joints discussed in this chapter pertains to frames which are used as the primary elements for resisting lateral forces. As indicated earlier, such frames must be capable of significant inelastic deformation without loss in strength. The preferred locations for inelastic deformation is in plastic hinge regions in beams, although a small amount of inelastic activity in the joints is acceptable and perhaps even desirable. At extreme loads plastic hinges can also develop at column bases. The stringent requirements for these ductile moment-resisting frames need not apply to frames of a structure intended only for carrying gravity loads.

In this section some suggestions for rudimentary calculations in the design of ductile moment-resisting frames are given. In order to satisfy these requirements, it is essential that the joints be strong enough to force plastic hinges to occur in the beams, or be able to deform plastically a moderate amount without loss of their own strength. An approach for satisfying some of these requirements for several elements of a joint are described in the following three subsections, which deal with flange connections, web connections, and panel zone design.

6.3.1 Flange Connections

There are several ways in which beams may be connected to a column. A number of different types of connections are discussed by Blodgett [1966] and Teal [1965,1976]. In seismic design the majority of beam flanges are connected directly to a column flange or column stiffeners (flange continuity plates) by full penetration welds made over back-up plates. Therefore, in this type of a design, a call-out for a full penetration groove weld with a back-up plate is all that is required for the design of the flange welds. This connection is considered to be sufficient for developing the full plastic moment M_p of the beam.

Welded moment connection plates of the type illustrated in Fig. 6.13 appear to be seldom used. However, if a bolted field assembly is decided upon, flange connections of the type shown for a small specimen in Fig. 6.22 [Popov and Pinkney, 1968b] are used. In this design, the flange connecting plates are shop-welded to the column with full penetration welds, and bolted in the field to the beam flanges. In designing this type of connection, great care must be exercised to assure the development of plastic hinges in beams before fracture could occur across a net section of the kind shown in Fig. 6.22. This requirement may be difficult to fulfill, in which case the beam flanges may have to be reinforced such that the plastic hinges would form outside the bolted area.

6.3.2 Web Connections

According to the ASCE Manual on Plastic Design in Steel [ASCE-WRC, 1971], the web connection must be designed to resist the shear due to factored gravity loads, considering the beam to be simply supported, and the effect of fully developed plastic moments at the ends of a beam (see Fig. 6.4). On this basis the maximum beam shear V_p can be expressed by the following equation:

$$V_p = V_g + \frac{2M_p}{L_c} \tag{6.1}$$

where V_g = shear due to gravity loads, assuming the beam to be simply supported;

M_p = plastic moment capacity of beam;

L_c = clear span of beam.

In applying this equation, the strength capacity of the web connection bolts or welds is used. Connections employing bolted webs are usually found to be less expensive than those employing welds for transmitting shear. Providing the welding sequence for the joint is carefully planned to minimize locked-in residual stresses; welded web connections develop somewhat greater ductility than those with bolted webs (see Fig. 6.9).

Since most of the current codes are written for combined gravity and seismic code loads on the basis of allowable stresses, it is usually necessary to check the designed web connection for compliance with such provisions.

6.3.3 Panel Zone Design

The panel zone design is not fully resolved. An approach believed to be conservative is outlined here; sometimes it may lead to excessive requirements. As was explained in Section 6.2.2, due to the high shear which develops in the panel zones, doubler plates may be required to prevent overstress and significant shear deformations. Strength considerations are primarily emphasized in this discussion, assuming that at low stresses the panel zone deformation is small. For drift considerations, the reader is referred elsewhere [Krawinkler *et al.*, 1971; Becker, 1975; Teal, 1975,1976]. Some background for this problem is given in Section 6.2.2. As indicated in that section, properly designed panel zones (Fig. 6.19) can experience inelastic shear deformations without causing a loss in strength in a subassemblage. On the other hand, excessive panel shear deformations (Fig. 6.18) can have a deleterious effect on the strength of a frame.

If the customary "strong column-weak girder" design approach (see Section 6.1.1) is adhered to, plastic hinges can form in the beams on both sides of a column, and it is reasonable to take the points of inflection in columns at their midheight. On this basis, the design panel shear V_p' (see Fig. 6.23) can be approximated as

$$V_p' = T_1 + C_2 - H$$

$$= \frac{M_{p1}}{0.95d_1} + \frac{M_{p2}}{0.95d_2} - \frac{M_{p1} + M_{p2}}{H_c} , \qquad (6.2)$$

where M_{p1} and M_{p2} = plastic moment capacities of beams, respectively, on the right and left of the column;

d_1 and d_2 = beam depths;

H_c = average story height at the joint.

The distance between the centroids of flanges is approximated by 0.95 times the beam depth. Since from the equilibrium conditions for a joint the beam moments are equilibrated by the moments in the column, the sum of the beam moments is divided by H_c to determine approximately the horizontal force H.

Except for the use of the unmodified beam depths, an expression similar to Eq. (6.2) is given in the ASCE Manual 41 [ASCE-AWS, 1971]. Similar expressions are also given in AISC Specifications Commentary on Part 2 [1980] without taking into account the beneficial effect of the force H.

The plastic shear capacity V_p of a column web, in the notation of this chapter, according to Eq. (2.5-1) of the AISC Specifications [1980] shall be proportioned such that

$$V_p \leqslant 0.55F_y t_w d_3 \quad , \tag{6.3}$$

where F_y = yield strength of steel;

t_w = thickness of column web;

d_3 = column depth (Fig. 6.23).

The coefficient 0.55 is the result of assuming the von Mises yield criterion in shear to be $F_y/\sqrt{3}$, and the use of $0.95d_3$ for the effective column depth d_3, i.e.,

$$\frac{F_y}{\sqrt{3}}(0.95d_3)t_w = 0.55\, F_y d_3 t_w \quad .$$

If V_p' given by Eq. (6.2) is smaller than or equal to V_p in Eq. (6.3), the column web is satisfactory. However, if V_p' is greater than V_p, a doubler plate is required. The thickness t_d of the doubler plate can be found simply by providing additional shear area in the panel zone for the difference between V_p' and V_p; i.e.,

$$t_d = \frac{V_p' - V_p}{0.55\, F_y d_3} \quad . \tag{6.4}$$

Minimum required welding should be used for attaching doubler plates to the column web; often it is more economical to select a larger size column rather than to incur additional fabrication costs by providing doubler plates.

The conservative approach for the design of doubler plates discussed above sometimes requires the use of very thick plates. Although the joints so designed are likely to remain elastic during a severe earthquake, the high residual stresses around the joint may create problems. For these reasons repeated efforts to justify the use of thinner doubler plates are being made. Some of the recent tests by Slutter [1981] have been indicated in Section 6.2.2; however, it is to be recalled that in these experiments no axial column loads were applied to the specimens, making the results questionable. Krawinkler [1978] attempted to justify the use of thinner doubler plates by including the contribution of column flanges; this appears to be a promising direction to pursue in the future.

The panel zone deformation problem usually does not arise in moment connections of beams to box columns or to the column webs, since two column plates are parallel to the beam web.

The design of the stiffener or continuity plates at a moment-resisting joint follows the procedures commonly employed in static design. However, one must recognize the possibility of complete reversal of moments that may reach the plastic capacity of the beams. Therefore, the flange forces should be calculated from the M_p's of the beams. Buckling of stiffeners must also be considered; the use of appropriate width-thickness ratios is essential for plastic design. A simplified buckling analysis of a plate supported on three sides and free along the fourth may be desirable in some cases. In all cases, the stiffener plates, in addition to being welded to the flanges, must also be welded to the column webs to provide for transfer of the beam flange forces. Connecting plates used in static design are unacceptable in ductile moment frames.

6.4 TYPICAL SEISMIC MOMENT CONNECTIONS

In this section, some widely used seismic moment connections are illustrated. These are of three different kinds. First, moment connections of beams-to-column flanges are given, then for connecting beams-to-column webs, and last, a related problem of flange and web column splices is brought in.

6.4.1 Moment Connections of Beams-to-Column Flanges

Since connection design and fabrication is a major part of steel design, it is important to select the best alternative for a specific job. The joints shown in Fig. 6.24 illustrate several types of moment connections of beams-to-column flanges, which have been successfully used on projects [AISC-SSEC, 1981]. These are arranged approximately according to their relative cost. The least expensive one, designated as CF-1, and shown in Fig. 6.24a, has the shear tab attached to the column with fillet welds and is bolted to the beam web with high strength bolts. The use of a full penetration weld for attaching the shear tab to the column (CF-2), as well as making an all-welded connection shown in Fig. 6.24b, increases the fabrication cost by a small amount (approximately 6 to 7 percent). (The bolts shown in Fig. 6.24b, as well as in Fig. 6.24c, are for erection only.) The cost of the connection shown in Fig. 6.24c is significantly higher (approximately 25 percent). The welding sequence must be carefully worked out for this joint, since a full penetration weld of the beam web to the column tends to introduce significant locked-in stresses due to restraint. For the joint shown in Fig. 6.24b, welding the web after welding the flanges minimizes the residual stresses.

The fabrication of the joint detailed in Fig. 6.24d is approximately double the cost of the one designated as CF-1 in Fig. 6.24a. Moreover, in this type of joint it is difficult to make an efficient connection between the moment plates and flanges. As stated in Section 6.3.1, unless the beam flanges are reinforced to force the formation of a plastic hinge outside the bolted area, there is the possibility of developing a fracture across a net section (Fig. 6.22). Nevertheless, if bolted field erection is required, satisfactory connections of this type between the moment plates and beam flanges can be achieved. Generally, high strength bolts are used throughout in this type of a joint.

In the joints shown in Fig. 6.24, no doubler plates are indicated. These can be proportioned using the procedure described in Section 6.3.3. As to the manner of detailing them, there is no unanimity (see Section 6.2.2). Some design engineers prefer fitting the doubler plates between the column flanges and the continuity (stiffener) plates and to require that welds develop the capacity of such plates, although there is some evidence that smaller welds can be used [Bertero *et al.*, 1973]. On the other hand, some designers prefer to use the detail shown in Fig. 6.20. In this detail, the doubler plate extends beyond the stiffeners. For the analysis of such joints without the effect of the axial column load, the reader is referred to an AISC-SSEC Report [1982].

In some instances, by virtue of the column flange thicknesses, it may not be necessary to provide column stiffener (continuity) plates; i.e., the condition shown in Fig. 4.2 is not serious either from the point of view of stresses or contribution to the story drift. Nevertheless, the shearing stresses in the panel zone may be excessive. In such cases, doubler plates without stiffeners have been used occasionally. It is customary to extend such doubler plates ($t_f + 2.5h$) above and below the beam framing into the column. In this relation, t_f is the thickness of the column flange and h is the clear distance between column flanges. The doubler plates are welded to the column to develop their shear capacity. No cyclic experiments into the inelastic range appear to be available for this kind of panel zone reinforcement.

Moment connections to box columns can be made in the same manner as illustrated in Fig. 6.24. Since, in such cases, thick column plates are parallel to the beam webs, no panel zone problems arise. However, it is necessary to have internal stiffeners or continuity plates opposite the flanges of the moment-resisting beams as shown in Fig. 6.25. Note the use of full penetration welds in connecting the stiffener plate to the column plates. The box beam detail shown appears to be favored by fabricators, although there are many other types.

6.4.2 Moment Connections of Beams-to-Column Webs

Several moment joints for connecting beams-to-column webs are shown in Fig. 6.26. The least expensive type is shown in Fig. 6.26a, where, in the field, the flanges are butt-welded and the webs are bolted using high strength bolts. The stiffener plates are shop-welded, both to the column flanges and to the column web. Although full penetration welds may be necessary in connecting these plates to the column, the less expensive fillet welds usually suffice for the vertical web plate. If an all-welded connection is required, the detail shown in Fig. 6.26b has been used in the past. Here the bolts are employed for erection purposes only. This joint is some 9 to 10 percent more expensive than the one with bolted web (Fig. 6.26a).

If it is decided to align the beam web with the web plate, the detail shown in Fig. 6.26c fulfills this requirement. Here an erection plate is used as a back-up plate to make a full penetration weld along the beam web. This connection is somewhat more costly than the bolted web connection (approximately 25 percent more).

It must be recognized that moment connections of beams-to-column webs for severe cyclically reversing loads are less reliable than connections to column flanges. Popov and Pinkney [1967] reported rather erratic behavior of such connections under cyclic loading. The ductility these connections attained generally was significantly smaller than that of connections made to column flanges. Often cracks propagated along the stiffener plates, as shown in Fig. 6.27, causing premature failure. These cracks initiated in the regions of high stress concentration at the juncture of the beam flange with the stiffener plate. Similar observations were made on large connections in monotonic tests by Rentschler *et al.* [1980].

To improve ductile behavior of the beam-to-column web connections, Driscoll and Beedle [1982] offered a number of useful suggestions. These are discussed in Chapter 5, Section 5.8. Based on some limited finite element analyses [Rentschler, 1979], it appears desirable to extend the continuity (stiffener) plates away from the column flanges (see Fig. 5.32). Based on this evidence, together with some recently completed Lehigh University tests by Driscoll, what is believed to be an improved detail is shown in Fig. 6.26d. Here the continuity plates are shown to extend beyond the column edge, and the web plates are lengthened appropriately.

Just as in box columns, no panel zone problem arises in the moment connections of beams-to-column webs, since the two column flanges are parallel to the beam webs.

In the event that field welding is to be avoided, a bolted connection similar to that shown in Fig. 6.24d can be designed. The problems that are encountered in this kind of a connection are the same as those for the connections of beams-to-column flanges.

6.4.3 Flange and Web Column Splices

Due to the lateral forces caused by an earthquake, column splices become subjected to both moment and shear. Generally, the column splices are located at a convenient distance above a floor level to facilitate construction and to keep them out of the regions of high moments occurring at the level of the floor beams. Two different column splice details are shown in Fig. 6.28. The detail illustrated in Fig. 6.28a is for a field welded assembly. By locating a splice in the region of a small bending moment, the calculated welds are often small, and AWS [1980] minimum partial penetration welds corresponding to the flange thicknesses are adopted. Such a practice can be justified only if the columns remain elastic.

Popov and Stephen [1977] have investigated a range of different size partial penetration welds in $W14 \times 320$ sections subjected to cyclic tension and compression. The results of these tests indicate that, from the strength point of view, welded splices in large column sections perform satisfactorily, even with minimum welds. However, partial penetration welded connections exhibit very little ductility, as may be seen from Fig. 6.29. Extrapolation of these results to situations with column bending leads to the same conclusion. This, coupled with uncertainty of the calculated forces at splices based on the current code provisions, indicates the need for conservative design.

The detail shown in Fig. 6.28b shows a typical column splice for field bolting. Again, a conservative design of such splices is necessary.

6.5 CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

This chapter deals with moment connections for regions of high seismic risk. The extent of experimental and analytical research on the behavior of seismic moment connections for such an environment is rather limited. Fortunately, one can draw on experience gained from the available information for monotonically applied loads. For this reason, Chapters 4 and 5 have direct relevance to seismic design. Of particular importance are the data on the amount of ductility that may be depended on for various kinds of connections. For seismic design the possibility of cyclic load reversal, however, must be recognized. The tension and compression regions of a joint may completely reverse.

Due to the great uncertainty of the forces which a structure may have to resist during a shake, complete reliance on the minimum code provisions is hazardous. Much judgment must be exercised in the design of a seismically resistant structure. An example of this kind of uncertainty was pointed out in Section 6.4.3 in connection with column splices.

The problem of a correct design for panel zones is not yet fully resolved. Thorough analytical and experimental evidence is lacking on a number of issues. The extent to which the thickness of the doubler plates can be reduced due to the effect of thick column flanges needs further study. In addition, the behavior of doubler plates without stiffeners must be investigated further. Both of these questions require full-size corroborative cyclic experiments.

The design of welded connections-to-column webs must be improved. The ongoing research on this problem at Lehigh University should provide valuable information.

6.6 ACKNOWLEDGEMENTS

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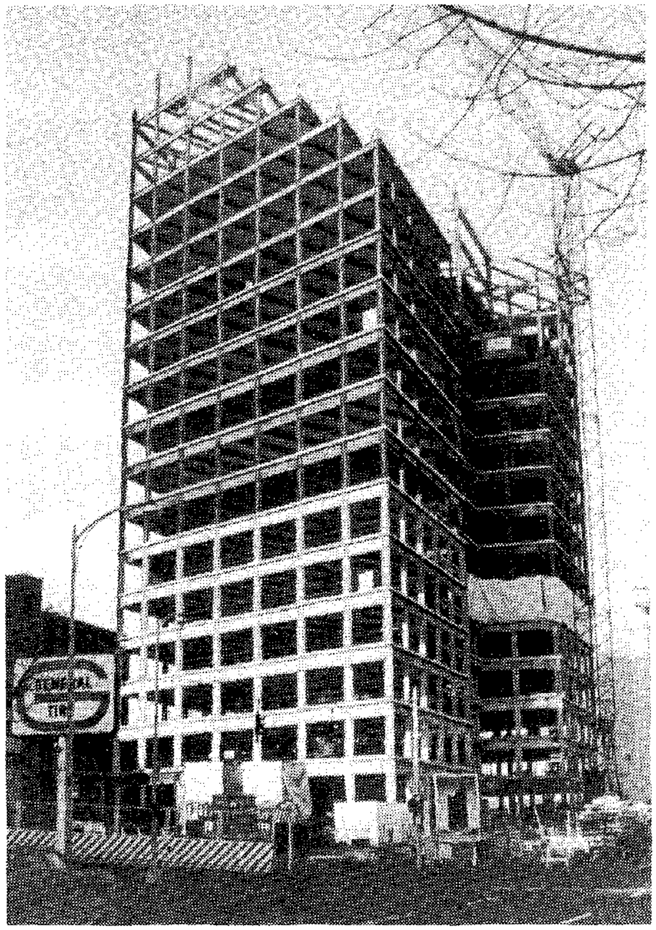


Fig. 6.1 Moment-Resisting Frame, Fourth and Blanchard Building, Seattle, Washington. (Photo courtesy United States Steel.)

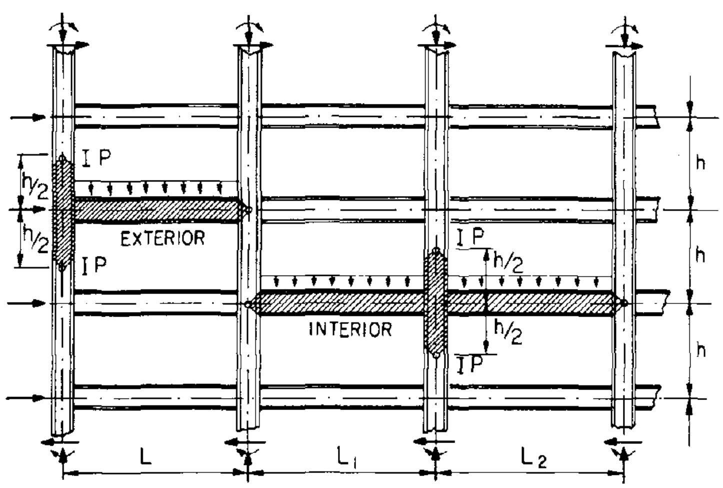


Fig. 6.2 A Portion of a Moment-Resisting Frame Illustrating Possible Test Subassemblages for Exterior and Interior Columns (Krawinkler, et al. 1971)

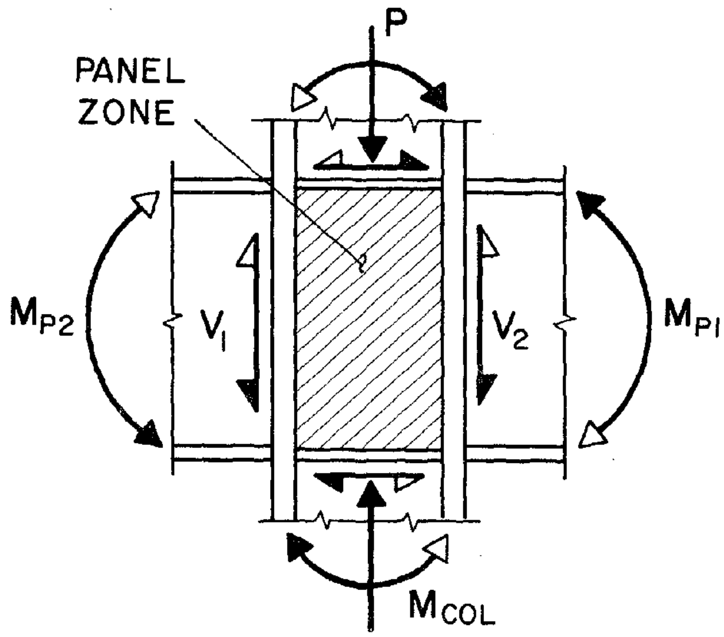


Fig. 6.3 Free-Body Diagram of a Moment-Resisting Joint.

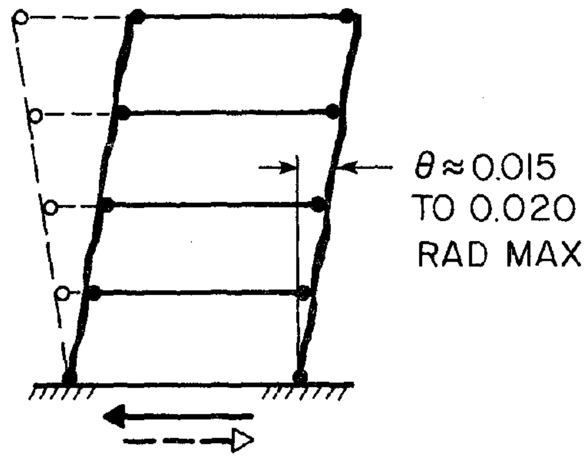


Fig. 6.4 Collapse Mechanism or Admissible Mechanism Motion for Determining Required Rotations at Joints.

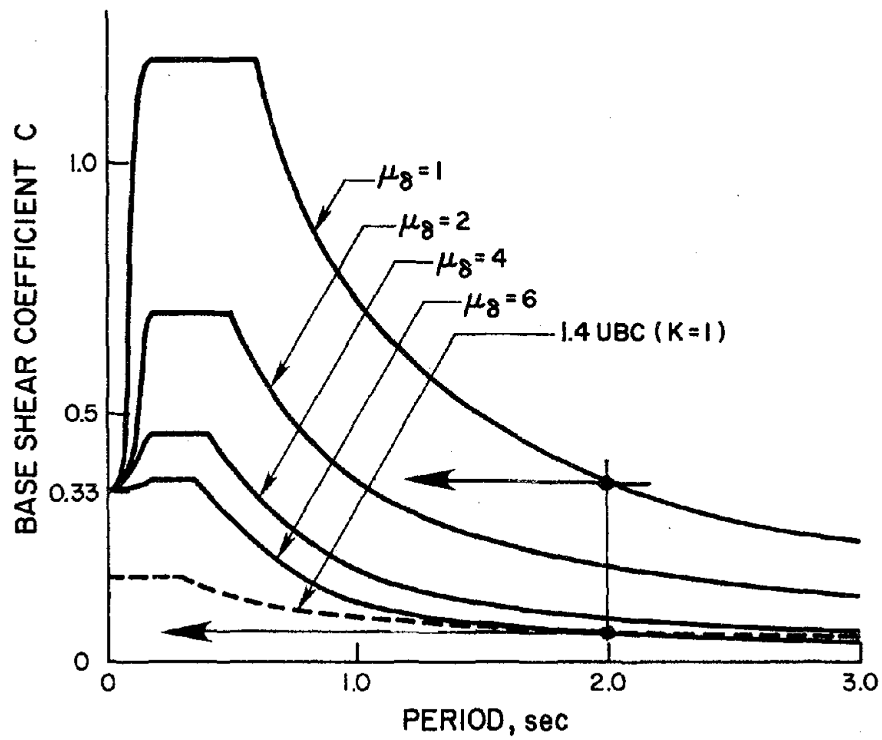


Fig. 6.5 Base Shear Coefficient Curves for Single Degree of Freedom Systems with Different Ductilities for a Severe Earthquake. Systems behave elastically for $\mu_\delta = 1$. In general, μ_δ is the ratio of maximum deflection to the deflection at yield.

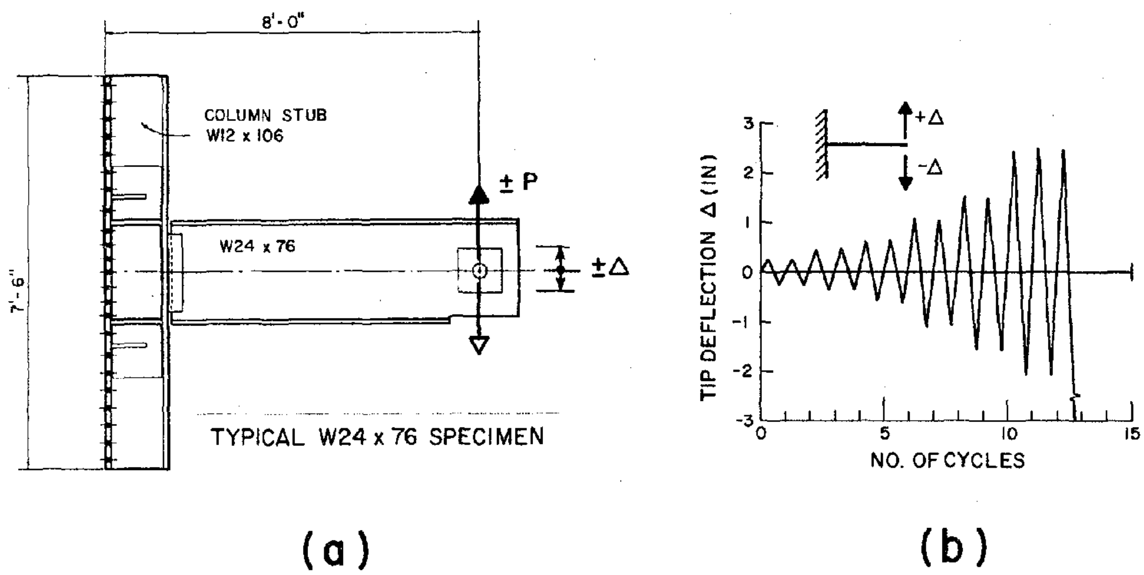


Fig. 6.6 (a) Full-Size Cantilever Specimen For Moment-Resisting Connection.
(b) Typical Cyclic Loading Sequence.

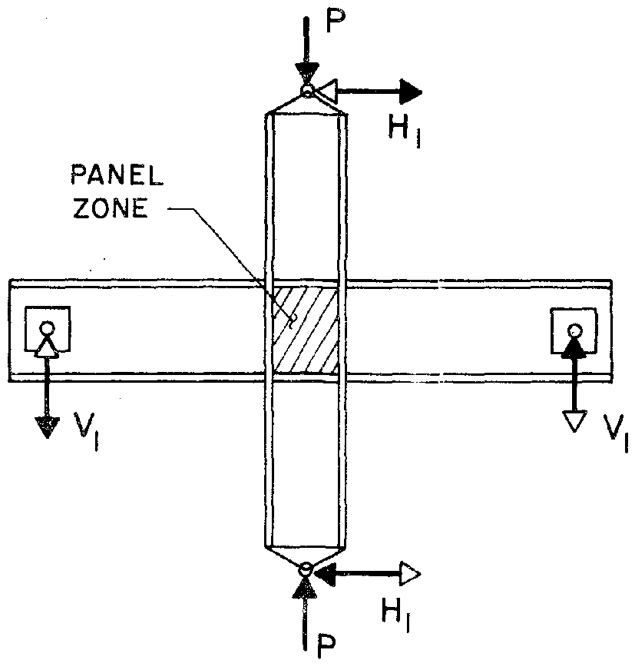


Fig. 6.7 Statically Determinate Specimen for Studying Cyclic Behavior of Panel Zone.

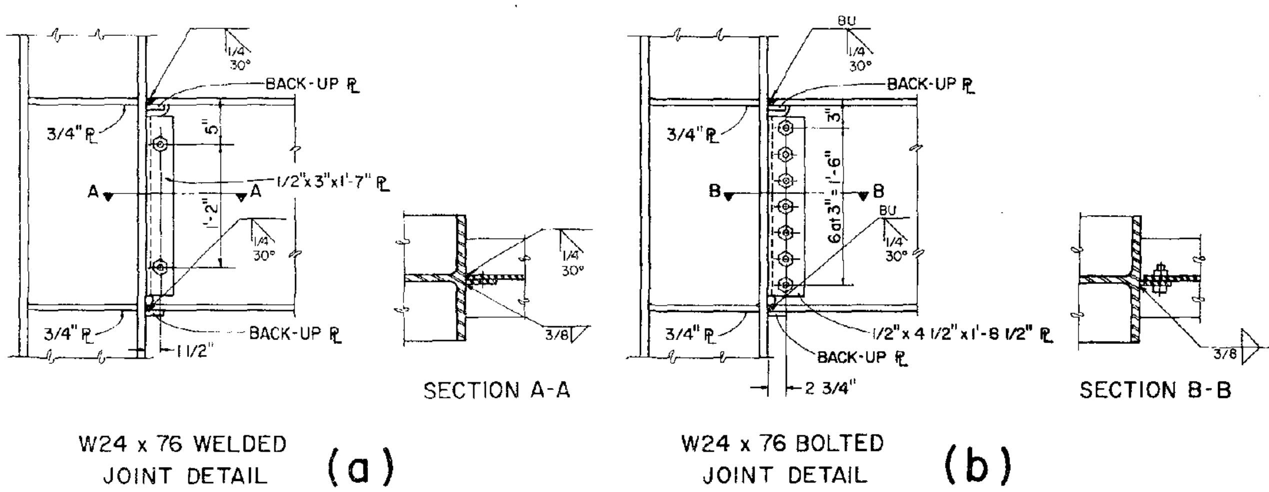
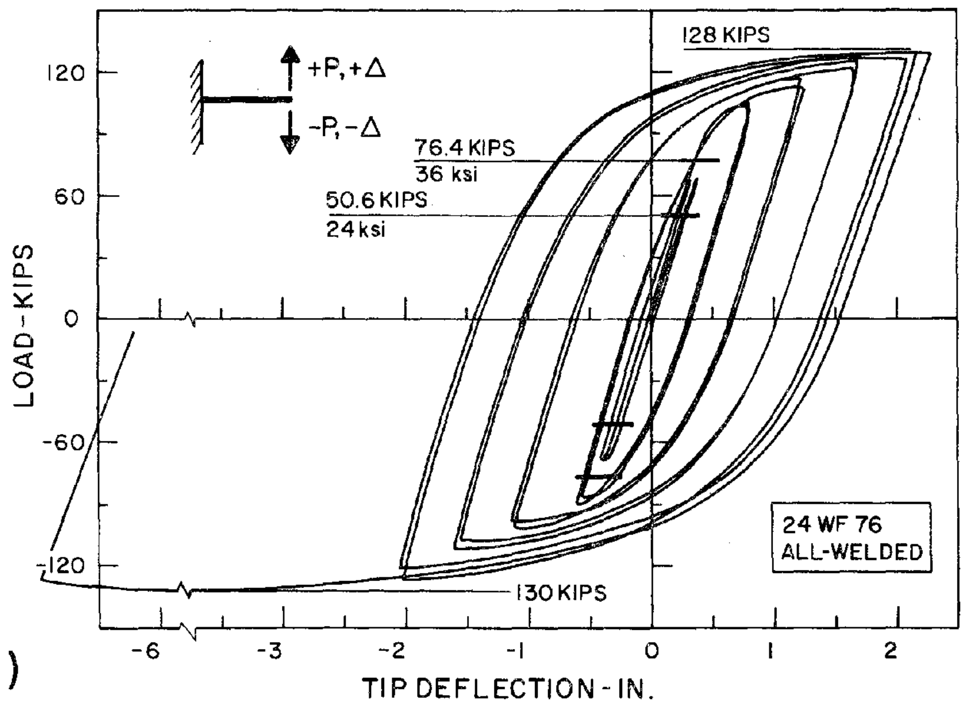
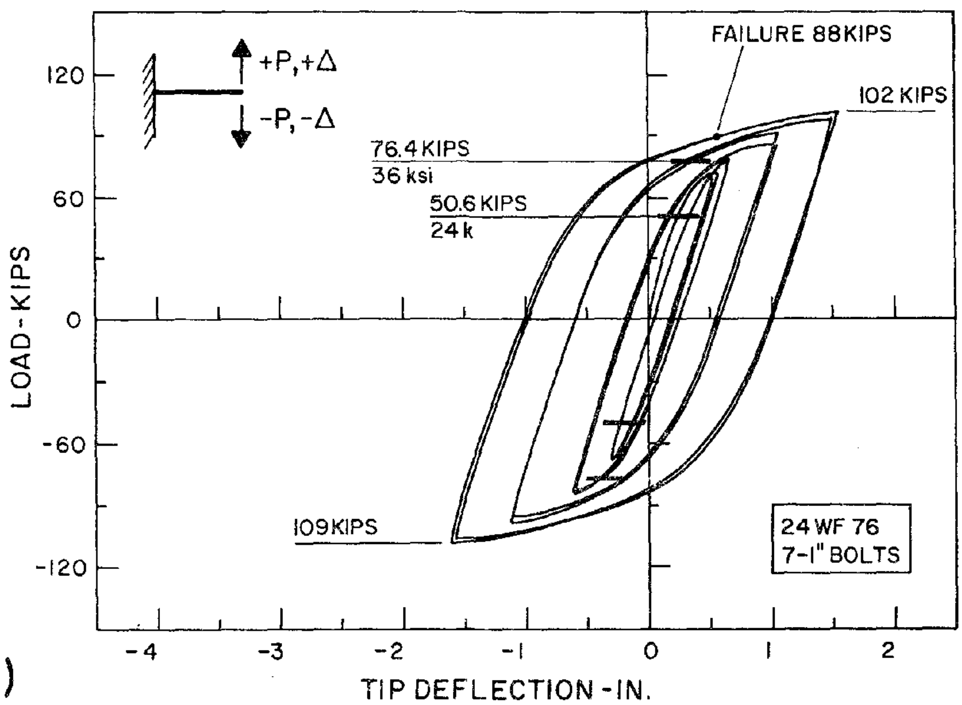


Fig. 6.8 (a) Joint Detail of Welded Moment-Resisting Connection with Two Erection Bolts.
(b) Joint Detail of Moment-Resisting Connection with Welded Flanges and Bolted Web.



(a)



(b)

Fig. 6.9 Load-Deflection Hysteretic Loops (a) for Welded Connection Specimen, and (b) for Welded Flanges-Bolted Web Specimen.

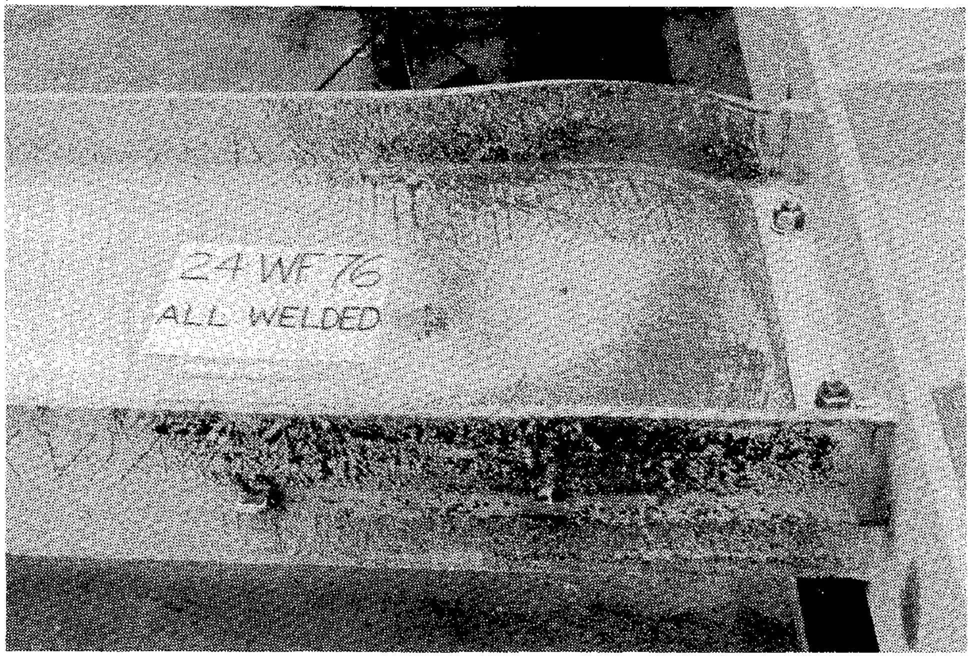


Fig. 6.10 Welded Connection Specimen After a Test Program Shown in Fig. 9 (a).

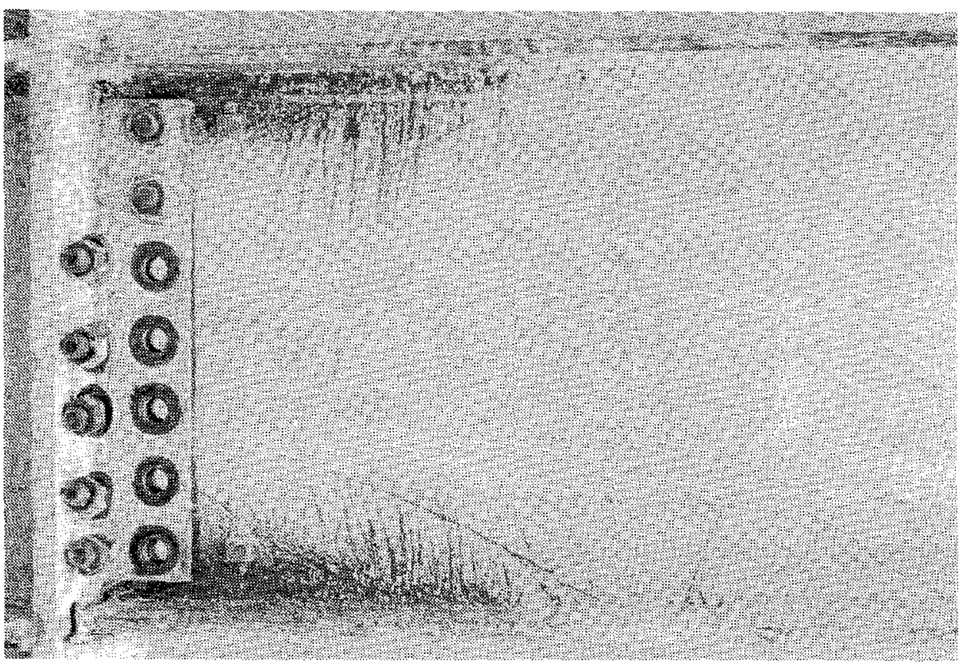


Fig. 6.11 Specimen with Bolted Web and Welded Flanges After a Test Program Shown in Fig. 9 (b).

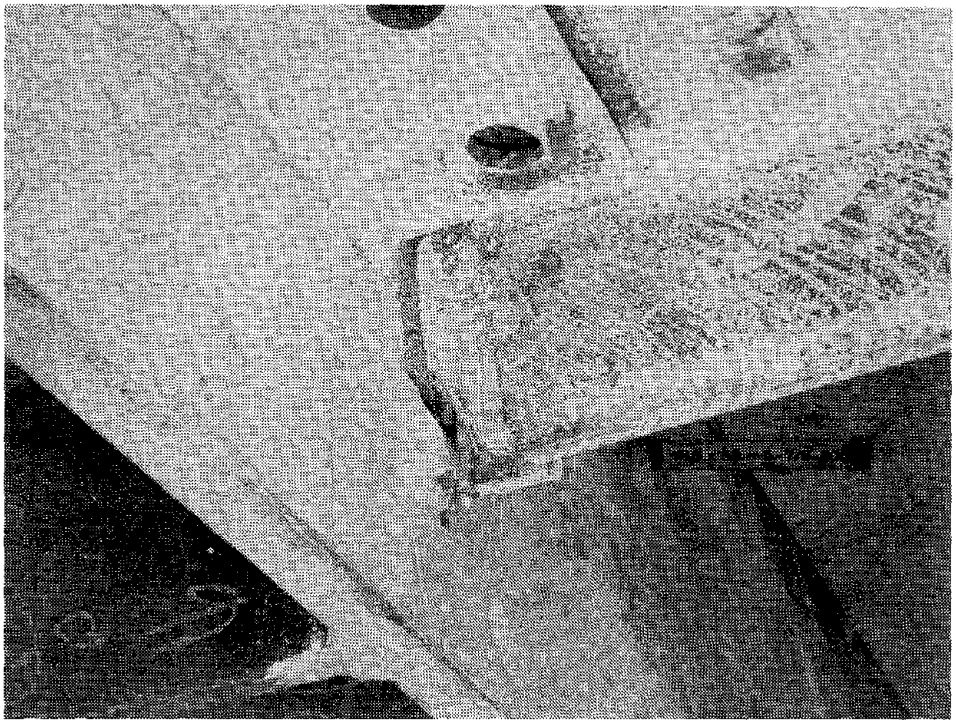


Fig. 6.12 Failure of W18 x 50 Specimen by Beam Flange Pull-out from Column Stub.

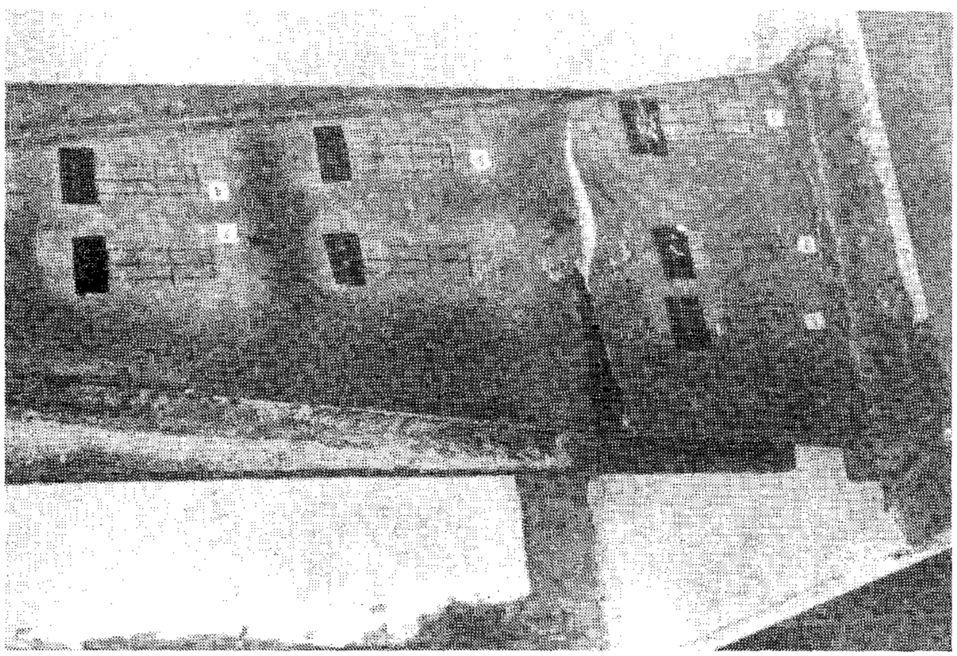


Fig. 6.13 Fracture of Moment Plate after 18 Severe Cyclic Load Reversals.

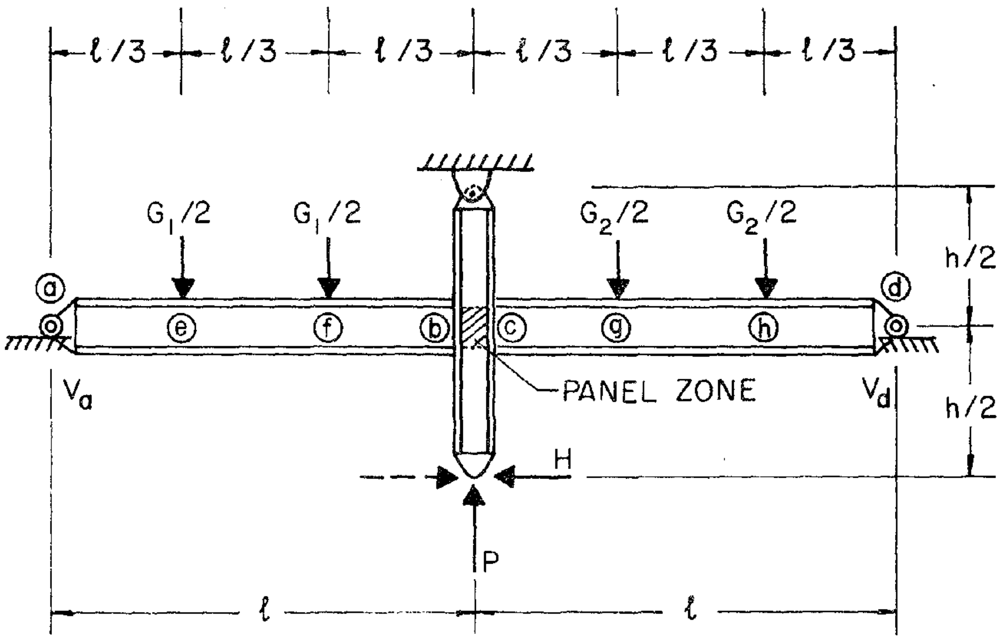


Fig. 6.14 Idealized Model of Interior Column Subassemblage. (Krawinkler, et al. 1971).

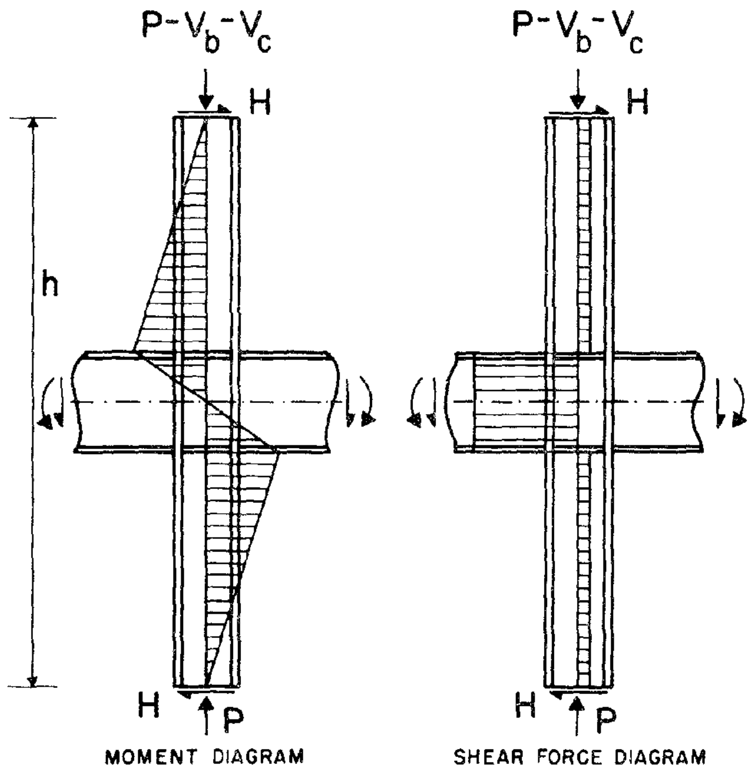


Fig. 6.15 Moment and Shear Diagrams for Interior Column. (Krawinkler, et al. 1971).

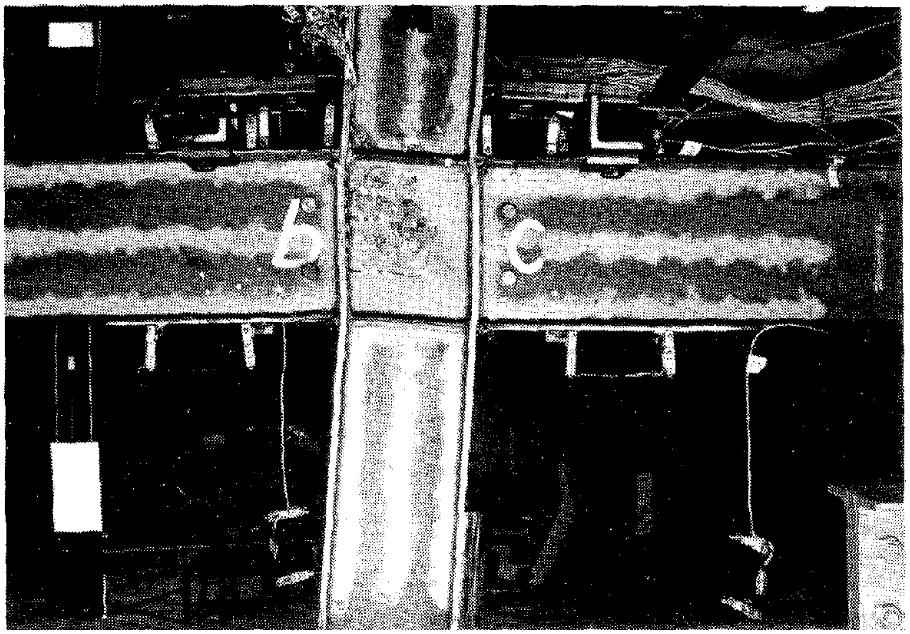


Fig. 6.16 Panel Zone Deformation in Advanced Stages of Testing. (Krawinkler, et al. 1971)

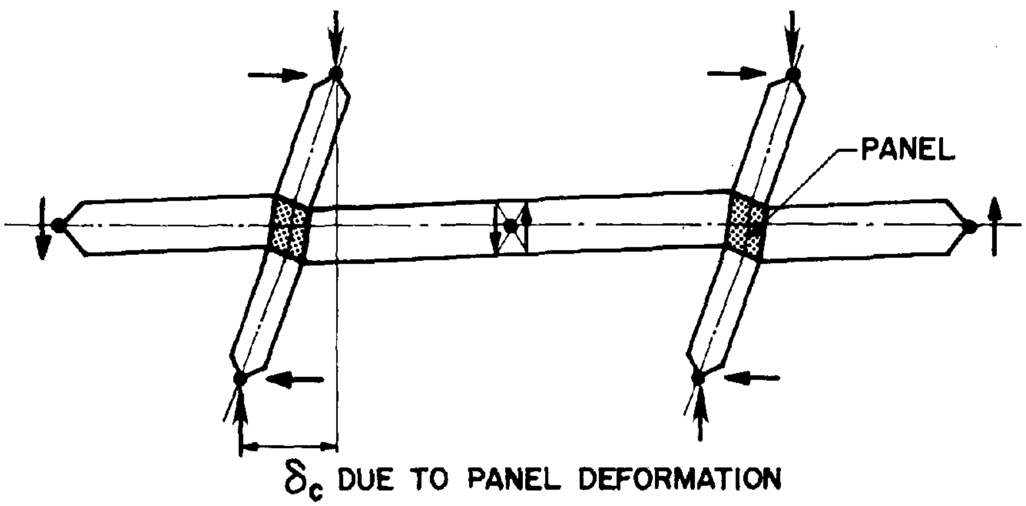


Fig. 6.17 Story Drift Due to Panel Zone Deformation.

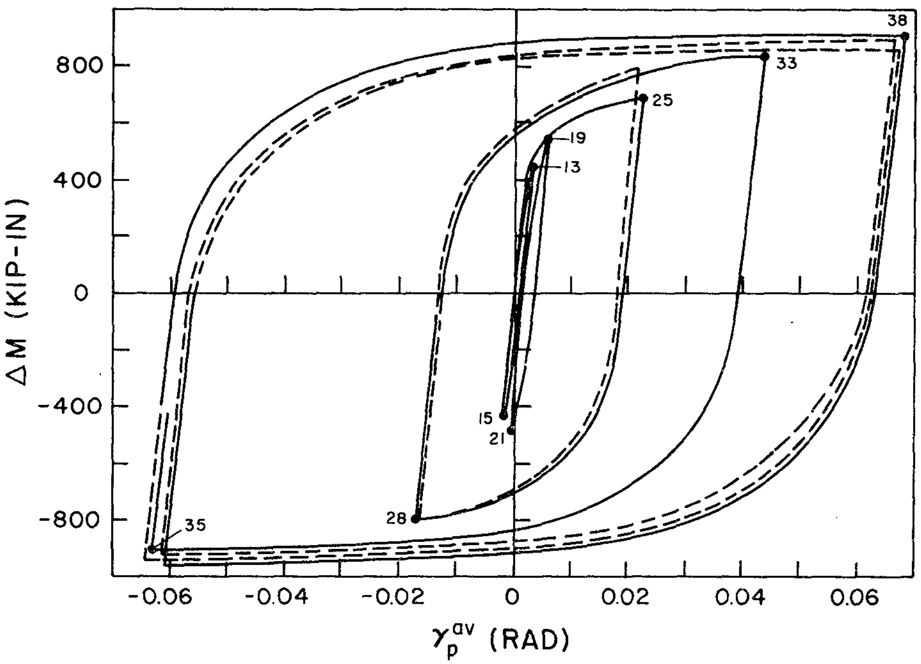


Fig. 6.18 Unbalanced Moment vs. Panel Zone Distortion for W8 x 24 Column. (Krawinkler, et al. 1971).

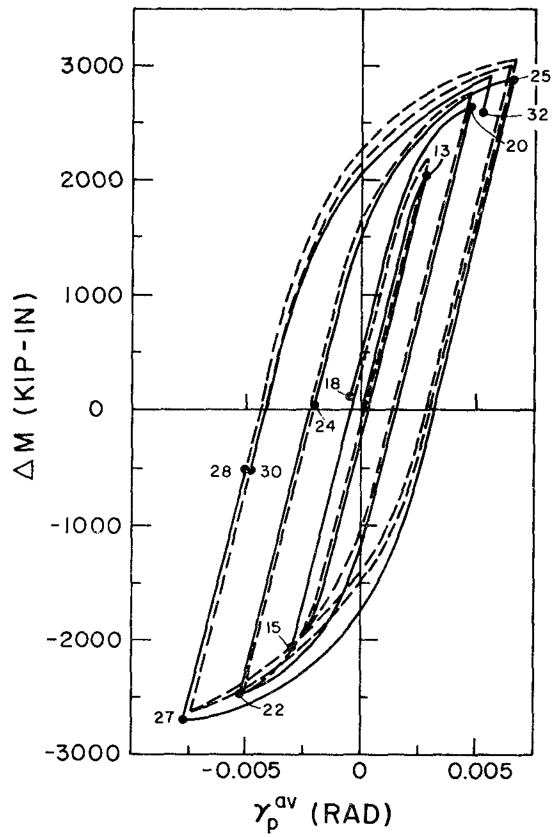


Fig. 6.19 Unbalanced Moment vs. Panel Zone Distortion for W8 x 67 Column. (Krawinkler, et al. 1971).

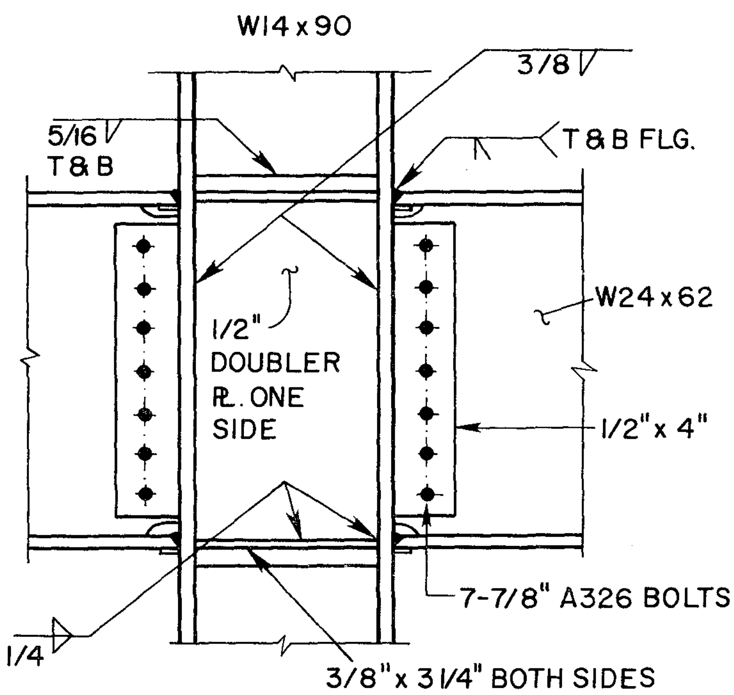


Fig. 6.20 Specimen 3 Joint Detail from Lehigh University Panel Zone Tests.

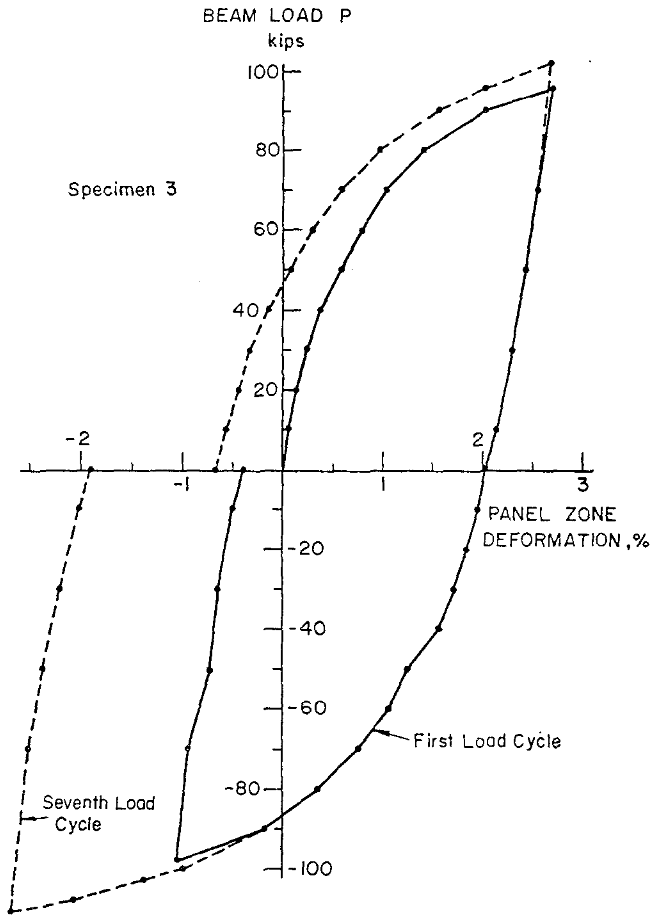


Fig. 6.21 Comparison of First and Seventh Cycle Load-Deformation Loops for Specimen 3 (Slutter, 1981)

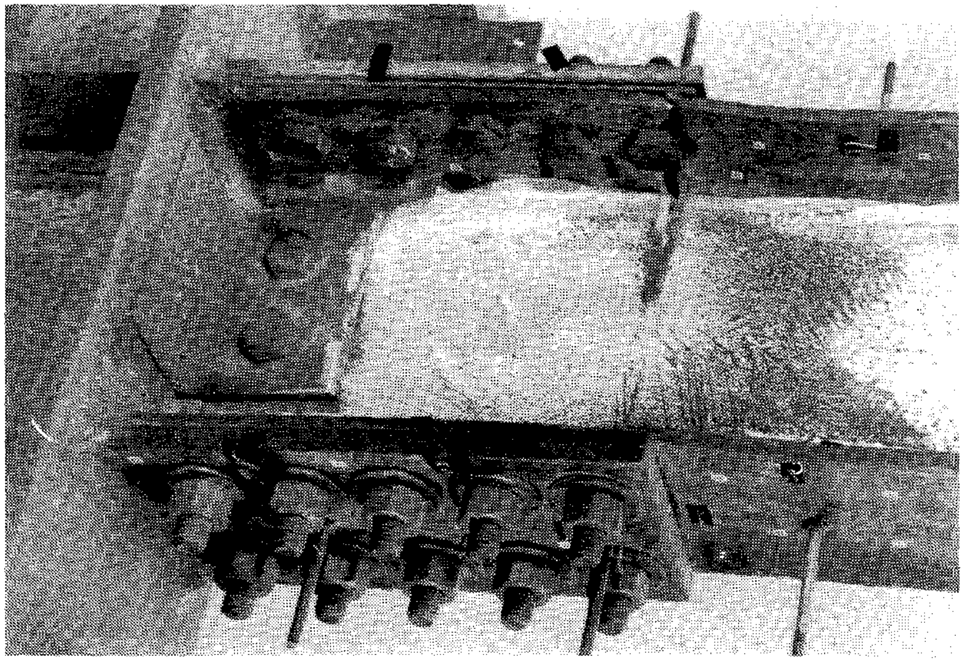


Fig. 6.22 Fracture Across Net Section for Bolted Connection in Cyclic Test. (Beam Flanges are Unreinforced.)

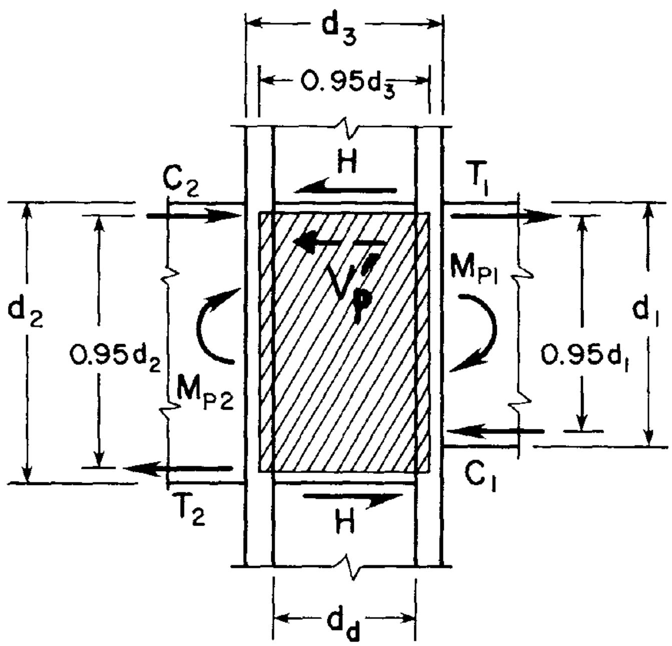


Fig. 6.23 Schematic Diagram for Determining Panel Zone Shear (not all forces are shown).

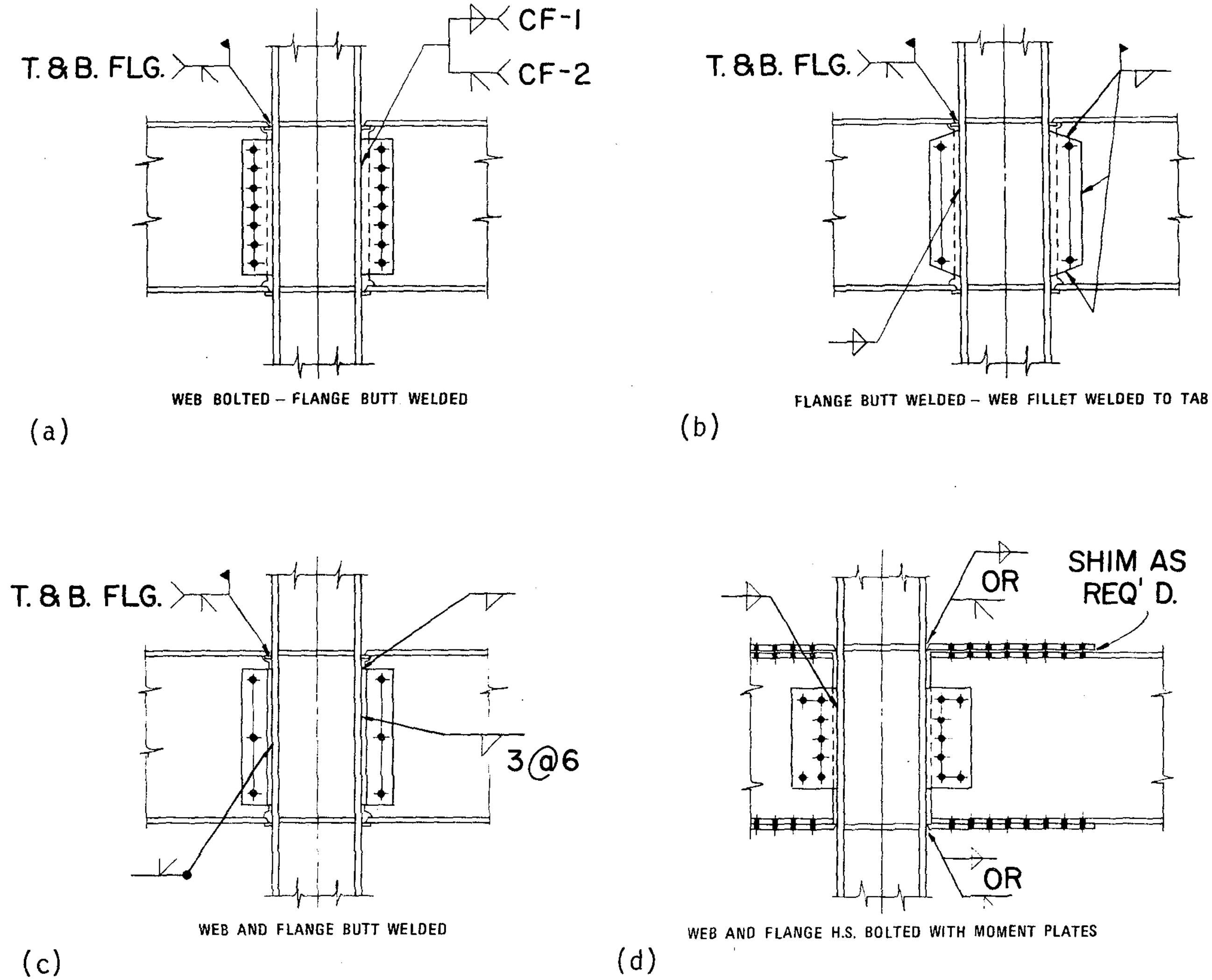


Fig. 6.24 Typical Moment Connections of Beams to Column Flanges (reproduced with permission of AISC-SSEC).

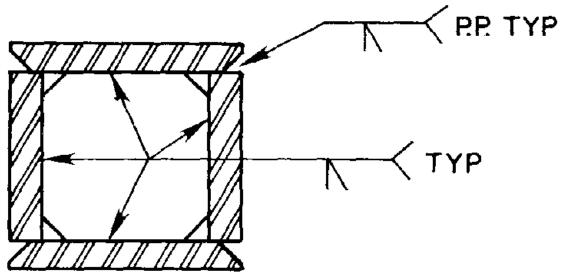


Fig. 6.25 Box Column Detail Showing Internal Stiffener Plate Opposite Flanges of Moment Beam Connections (reproduced with permission of AISC-SSEC).

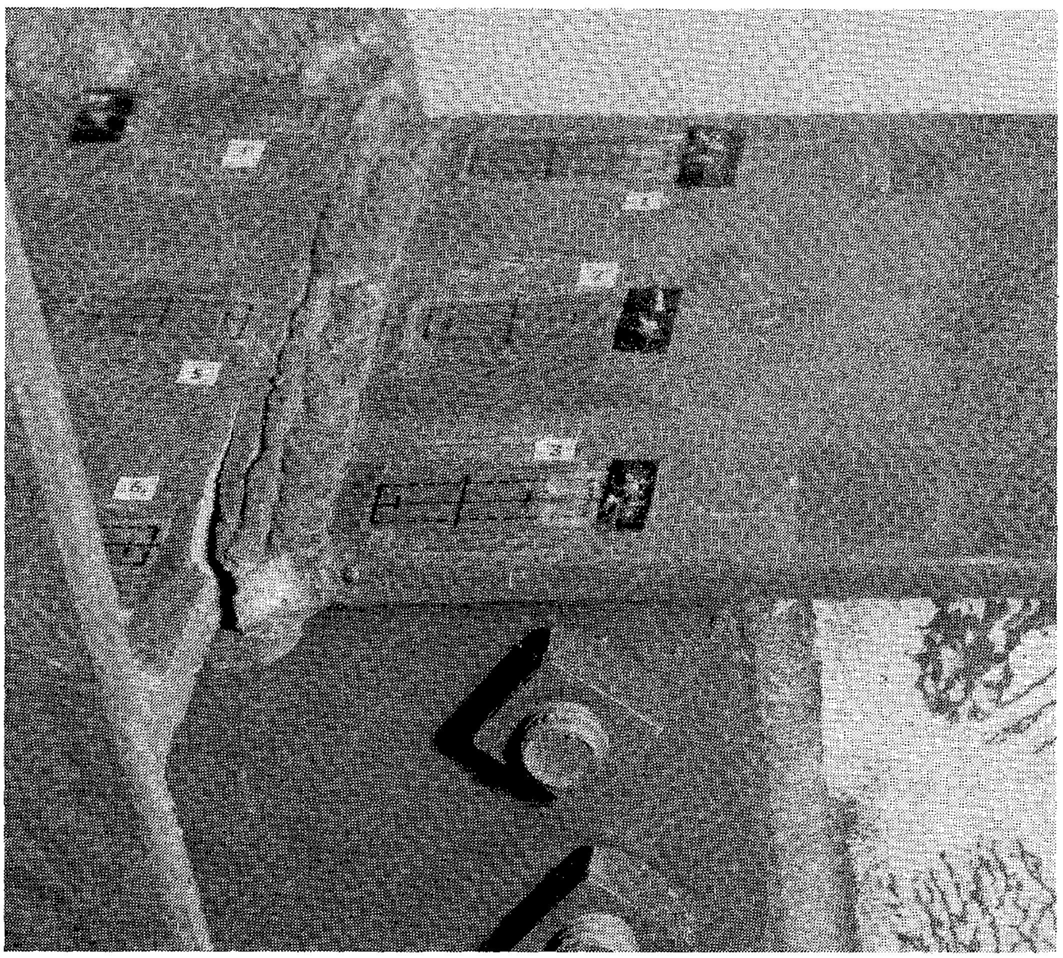


Fig. 6.27 Fracture in a Cyclic Test of Column Stiffener Plate for Beam to Column Web Moment Connection.

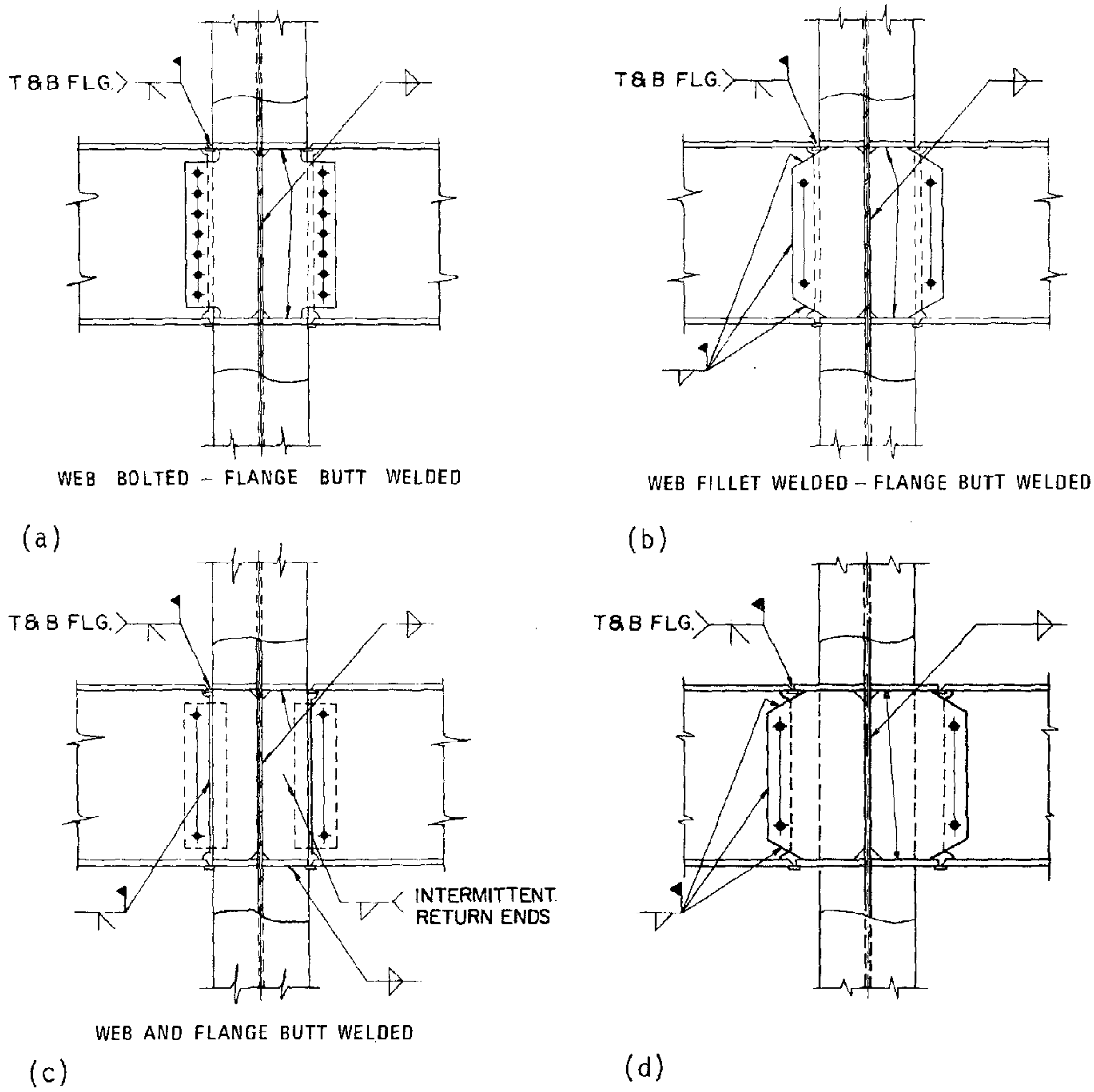


Fig. 6.26 Typical Moment Connections of Beams to Column Webs (details a, b, and c reproduced with permission of AISC-SSEC).

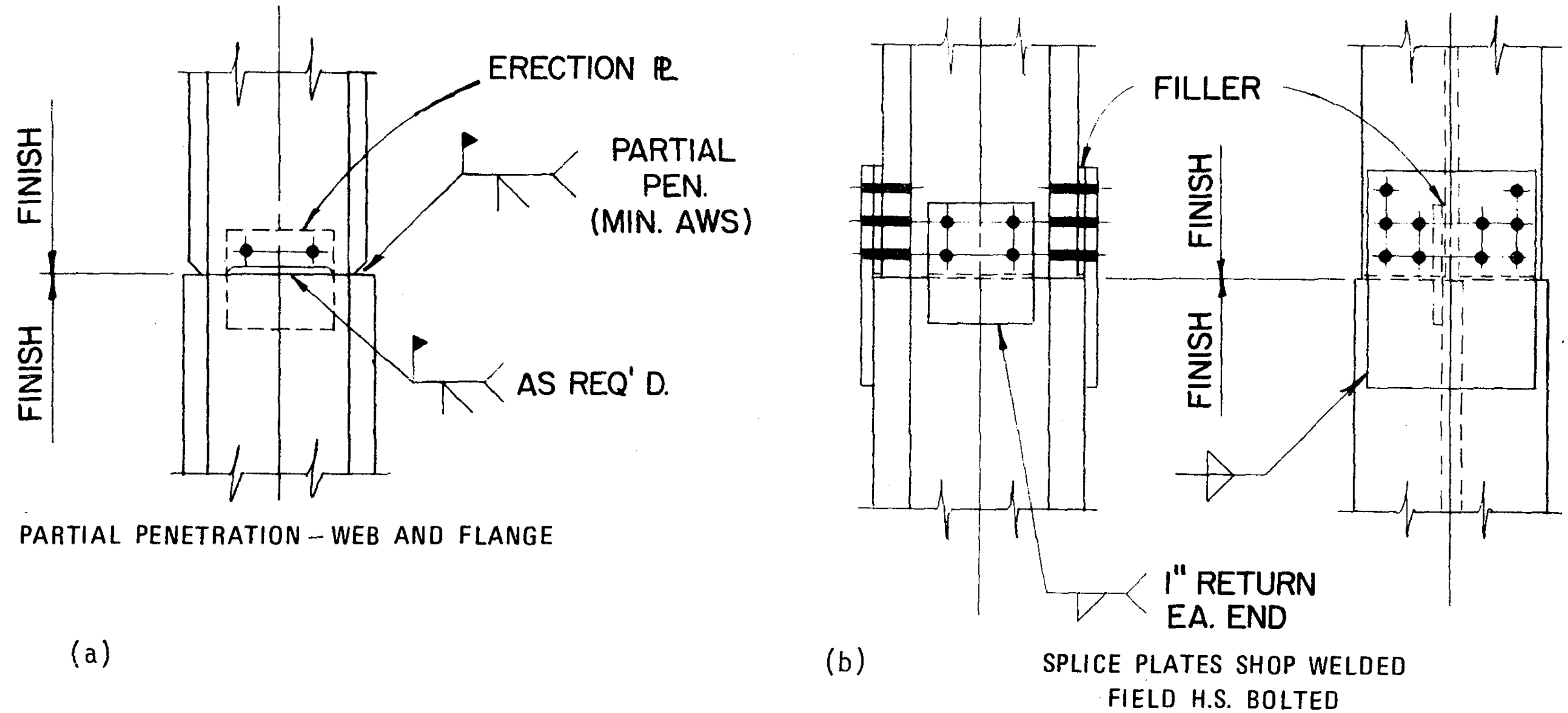


Fig. 6.28 Typical Flange and Web Column Splices (reproduced with permission of AISC-SSEC).

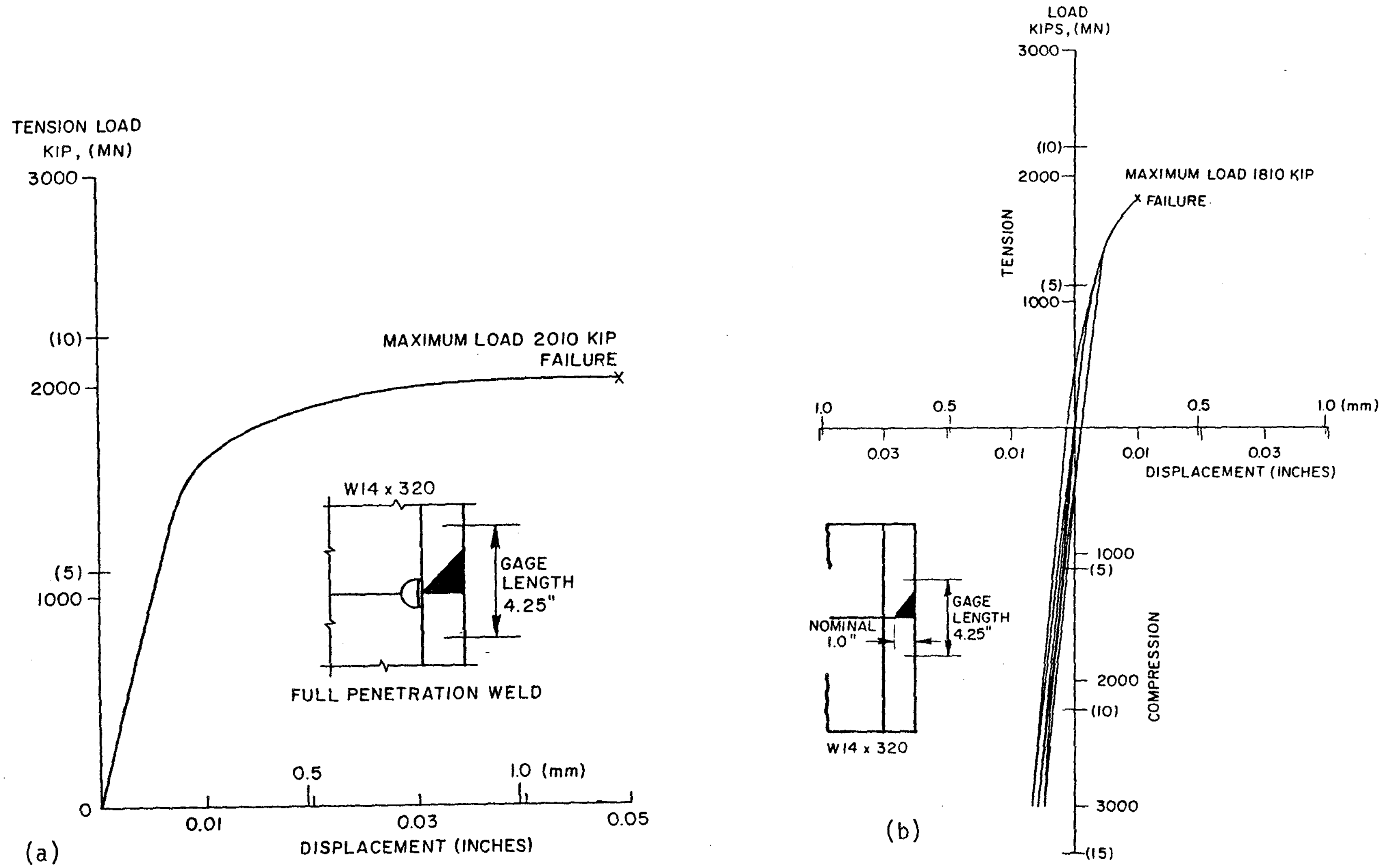


Fig. 6.29. Ductility Comparisons Across Welded Joints of Large Specimens in Tension.
(a) Full Penetration Flange Welds. (b) Partial Penetration Flange Welds.

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NOTE: Numbers in parentheses are Accession Numbers assigned by the National Technical Information Service; these are followed by a price code. Copies of the reports may be ordered from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia, 22161. Accession Numbers should be quoted on orders for reports (PB --- ---) and remittance must accompany each order. Reports without this information were not available at time of printing. The complete list of EERC reports (from EERC 67-1) is available upon request from the Earthquake Engineering Research Center, University of California, Berkeley, 47th Street and Hoffman Boulevard, Richmond, California 94804.

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