6. POST-EARTHQUAKE REPAIR AND MODIFICATION

As used in these Interim Guidelines, repair means restoration of the strength, stiffness and deformation capacity of structural elements that have been damaged or have construction defects. Modification means actions taken to enhance the strength, stiffness or deformation capacity of either damaged or undamaged elements, or of the structure as a whole.

Based on the observed behavior of actual buildings in the Northridge Earthquake, as well as recent test data, WSMF structures constructed with the typical pre-Northridge detailing and construction practice prevalent prior to the Northridge Earthquake do not have the same deformation capacity they were presumed to possess at the time of their design. The seismic risk associated with these structures is higher than typically judged as acceptable for buildings of new construction. When these buildings are damaged or have excessive construction defects, the risk is higher.

Based on limited testing, it appears that the repair recommendations contained in this Chapter can be effective in restoring a building's pre-earthquake condition. This does not imply, however, that the repaired building will be an acceptable seismic risk. As a minimum, it should be assumed that buildings that are repaired, but not modified, can sustain similar and possibly more severe damage in future earthquakes than they did in the present event. If this is unacceptable, either to the owner or the building official, then the building should be modified to provide improved future performance. Modification can consist of local reinforcement of individual moment connections as well as alteration of the basic lateral-force-resisting characteristics of the structure through addition of braced frames, shear walls, base isolation, energy dissipation devices, etc.

6.1 Scope

This section provides interim guidelines for structural repair of earthquake damage and modification of structures to improve future earthquake performance. *Repair* constitutes any measure(s) taken to restore earthquake damaged joints, connections, elements of the building, or the building as whole, to their original strength, stiffness and deformation capacity. It does not include routine correction of non-conforming conditions during original fabrication. Interim Guidelines for acceptable methods of repair are provided in Sections 6.2 through 6.5 below. These Interim Guidelines are not intended to be used for the routine repairs of non-conformance commonly encountered in fabrication and erection work. Industry standard practices are acceptable for such repairs.

Work that increases structural stiffness or strength of an element or the structure as a whole by more than 5% is classified as *modification*. Guidelines for some methods of modification are contained in Section 6.6.

6.2 Shoring

6.2.1 Investigation

The structural engineer responsible for designing any damage repair should investigate the entire building for imminent collapse or life safety hazard conditions, regardless of joint considerations. Such conditions should be shored prior to commencement of any repairs.

Commentary: In projects relating to construction of new buildings, it is common practice to delegate all responsibility for temporary shoring and bracing of the structure to the contractor. Such practice may not be appropriate for severely damaged buildings. The structural engineer should work closely with the contractor to define shoring and bracing requirements. Some structural engineers may wish to perform the design of temporary bracing systems. If the contractor performs such design, the structural engineer should review the designs for adequacy and potential effects on the structure prior to implementation.

6.2.2 Special Requirements.

Conditions which may become collapse or life safety hazards during the repair operations should be considered in the development of repair details and specifications, whether they involve the connection area directly or indirectly. These conditions should be brought to the attention of the contractor by the structural engineer, and adequate means of shoring these conditions should be provided. Consideration should be given to sequencing of repair procedures for proper design of any required shoring. For column repair details that require removal of 20% or more of the damaged cross section, consideration should be given to the need for shoring to prevent overstress of elements due to redistribution of loads.

Commentary: In general, contractors will not have adequate resources to define when such shoring is necessary. Therefore, the Contract Documents should clearly indicate when and where shoring is required. Design of this shoring may be provided by the structural engineer, or the contractor may submit a shoring design to the structural engineer for review.

6.3 Repair Details

The scope of repair work should be shown on drawings and specifications prepared by a structural engineer. The drawings should clearly indicate the areas requiring repair, as well as all repair procedures, details, and specifications necessary to properly implement the proposed repair. Sample repair details for various types of damage are included in these Interim Guidelines, for reference, only.

Commentary: Examples of repair details are provided for some classes of damage, based on previous repairs performed in the field for specific projects. Limited testing indicates these repair methods can be effective. Details are not *complete in all respects and should not be used verbatim, as construction documents. Many repairs will require the application of more than one operation, as represented by a given detail. The sample details indicated may not be directly applicable to specific repair conditions. The structural engineer is cautioned to thoroughly review the conditions at each damaged element, connection or joint, and to determine the applicability and suitability of these details based on sound structural engineering judgment, prior to employing them on projects.*

6.3.1 Approach

Based on the nature and extent of damage several alternative approaches to repair should be considered. Repair approaches may include, but should not be limited to:

a) replacement of portions of base metal (i.e. column and beam section),

b) replacement of connection elements,

c) replacement of connection weld, or

d) repairs to portions of any of the aforementioned components.

Any or all of these techniques may be appropriate. The approach(s) used should consider adjacent structural components which may be affected by the repair or the effects of the repair.

Where base material is to be removed and replaced with plates, clear direction should be given to orient the plates with the direction of rolling of the plate parallel to the direction of application of major axial loads to be resisted by the plate.

6.3.2 Weld Fractures - Type W Damage

All fractures and rejectable defects found in weld material, either between girder and column or between connection element and structural member, should have sufficient material removed to completely eliminate any discontinuity or defect. NDT should be used to determine the extent of fracture or defect and sufficient material should be removed to encompass the damaged area. It is suggested that material removal extend 2 inches beyond the apparent end of the fracture or defect. Simple fillet welds may be repaired by backgouging to eliminate unsound weld material and replacing the damaged weld with sound material. Complete joint penetration (*CJP*) welds fractured through the full thickness should be replaced with sound material deposited in strict accordance with the Welding Procedure Specification (WPS) and project specifications. The use of weld dams on new welds is prohibited. Weld backing (backup bars), existing dams, and weld tabs should be removed from all welds that are being repaired. After backing is removed, the root should be backgouged to sound material, rewelded and a reinforcing fillet added.

The structural engineer is cautioned to observe the provisions of AISC regarding intermixing of weld metals deposited by different weld processes (see AISC LRFD Manual of Steel

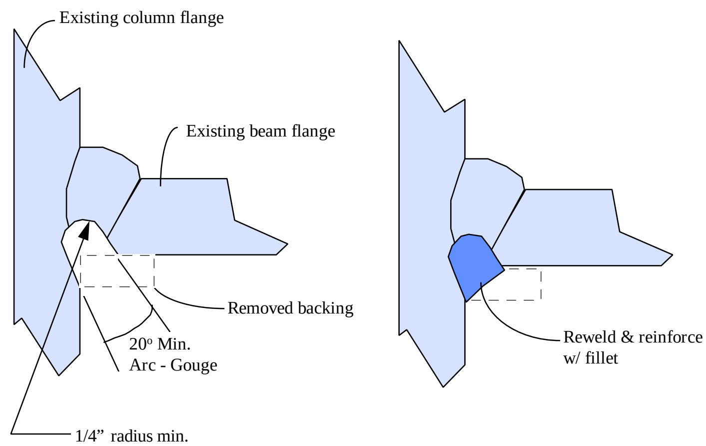
Construction, second edition, page 6-77, and AISC ASD Steel Construction Manual, ninth edition, page 5-69). As an example, E7018 stick electrodes should not be used to weld over self-shielded flux cored arc welding deposits. Removed weld material from fractures not penetrating the full weld thickness should be replaced in the same manner as full thickness fractures. For other types of W damage, existing backing, end dams, and weld tabs should also be removed in a like manner to *CJP* weld replacement. Table 6-1 provides an index to suggested repair details for type W damage.

Table 6-1 - Reference Details for Type W Damage

Damage Class	Figure
W1a, W1b	Figure 6-1, Figure 6-2
W2	Figure 6-3
W3	Figure 6-3
W4	Figure 6-3
W5	Figure 6-3

Commentary: FCAW-ss utilizes approximately 1-2% aluminum in the electrode to protect the weld from mixing with atmospheric nitrogen and oxygen. By itself, aluminum can reduce the toughness and ductility of weld metal. The design of FCAW-ss electrodes requires the balance of other alloys in the deposit to compensate for the effects of aluminum. Other welding processes rely on fluxes and/or gasses to protect the weld metal from the atmosphere, relieving them of any requirement to contain aluminum or other elements that offset the effects of aluminum. If the original weld that is being repaired consists of FCAW-ss and subsequent repair welds are made with SMAW (stick) using E7018, for example, the SMA arc will penetrate into the FCAW-ss deposit, resulting in the addition of some aluminum into the SMAW deposit. The notch toughness and/or ductility of the resultant weld metal may be substantially reduced as compared to pure E7018 weld metal, based on the depth of penetration into the FCAW-ss material.

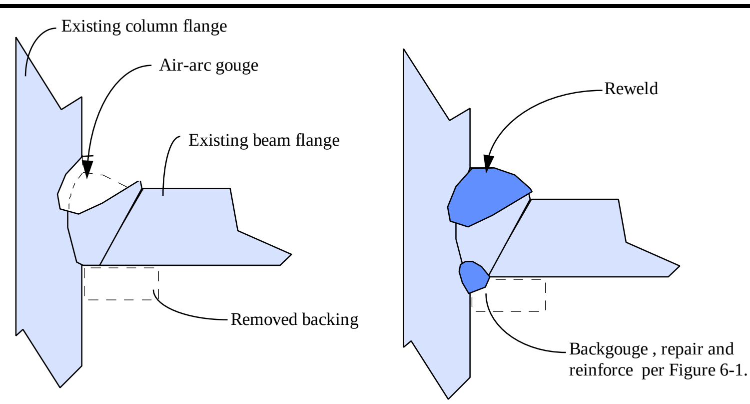
Various types of FCAW-ss electrodes may be mixed one with the other without potentially harmful effect. Further, FCAW-ss may be used to weld over other types of weld deposits without potentially harmful interaction. The structural engineer could specify all repairs on FCAW-ss deposits be made with FCAW-ss. Alternately, intermixing of FCAW-ss and other processes could be permitted provided the subsequent composition is demonstrated to meet material specification requirements.



Notes:

1 Remove existing backing.
2 Taper the depth of grinding or air arc gouging at each end to the face of flange with a minimum 2:1 (horizontal/vertical) taper. Provide a minimum root radius of 1/4.”
3 Grind all surfaces on which weld metal will be deposited. Surfaces should be smooth, uniform and free from fins, tears, fractures and other discontinuities which would adversely affect weld strength.
4 A fillet weld should be applied to reinforce the joint. The size of the reinforcing fillet should be equal to 1/4 of the beam flange thickness, but not less than 1/4.” It need nor be more than 3/8.”
5 On joints to be repaired, remove all remaining weld tabs and excess weld metal beyond the length of the joint and grind smooth. Imperfection less than 1/16" should be removed by grinding. Repair as necessary.

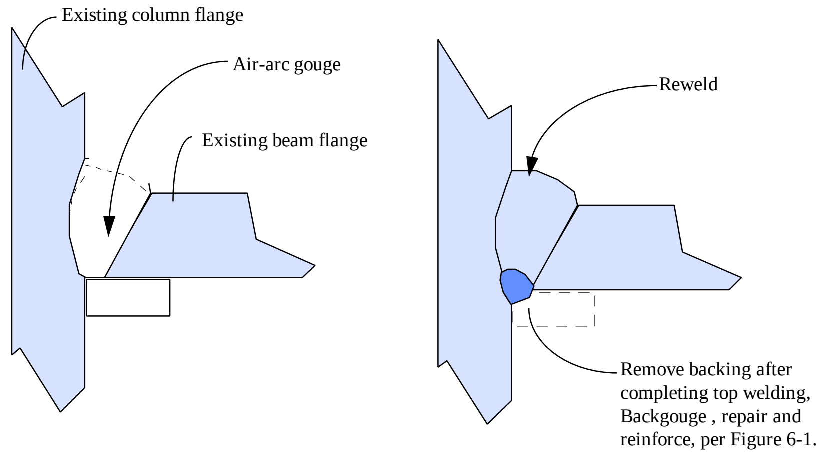
Figure 6-1 - Gouge & Re-weld of Root Defect or Damage - W1



Notes:

1. Remove the entire fracture plus 2” of sound metal beyond each end.
2. For additional notes, refer to Figure 6-1

Figure 6-2 - Gouge & Re-weld of Fractured Weld - W1



For notes see Figure 6-1 and 6-2.

Figure 6-3 - Backgouge and Reweld repair

6.3.3 Column fractures - Type C1 - C5 and P1 - P6

Any column fracture observable with the naked eye or found by NDT and classified as rejectable in accordance with the AWS D1.1 criteria for Static Structures should be repaired. Repairs should include removing the fracture such that no sign of rejectable discontinuity or defect within a six (6) inch radius around the fracture remains. Removal should include eliminating any zones of fracture propagation, with a minimum of heat used in the removal process. Following removal of material, MT and PT should be used to confirm that all fractured material has been removed. Repairs of removed material may consist of replacement of portions of column section, build-up with weld material where small portions of column were removed, or local replacement of removed base metal with weld material. Procedures of weld fracture repair should be applied to limit the heat affected area and to provide adequate ductility to the repaired joint. Tables 6-2 and 6-3 indicate representative details for these repairs. In many cases, it may be necessary to remove a portion of the girder framing to a column, in order to attain necessary access to perform repair work, per Figure 6-4. Refer to Section 6.3.5 for repair of girders.

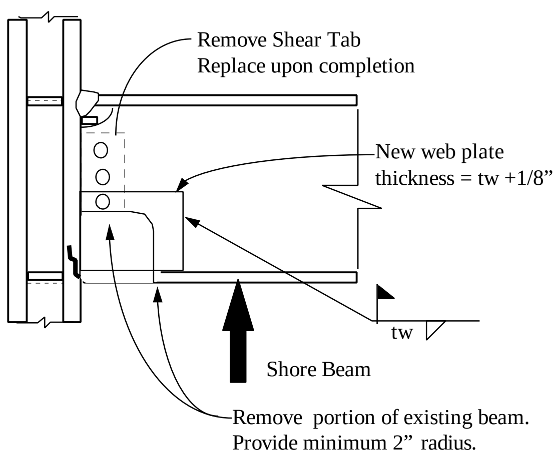
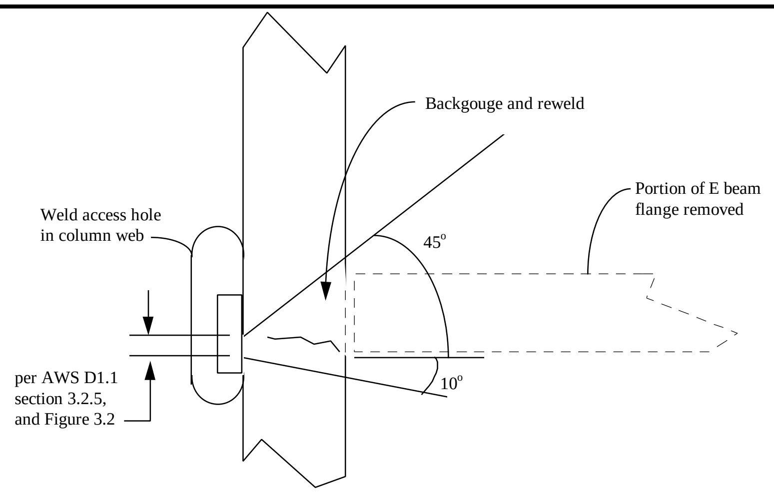


Figure 6-4 - Temporary Removal of Beam Section for Access

When the size of divot (type C2) or transverse column fractures (types C1, C3, C4) dictate a total cut-out of a portion of a column flange or web (types P6, P7), the replacement material should be ultrasonically tested in accordance with ASTM A578-92, "Straight-Beam Ultrasonic Examination of Plane and Clad Steel Plates for Special Applications," in conjunction with AWS K6.3 "Shearwave Calibration." Acceptance criteria should be that of Level III. The replacement material should be aligned with the rolling direction matching that of the column.

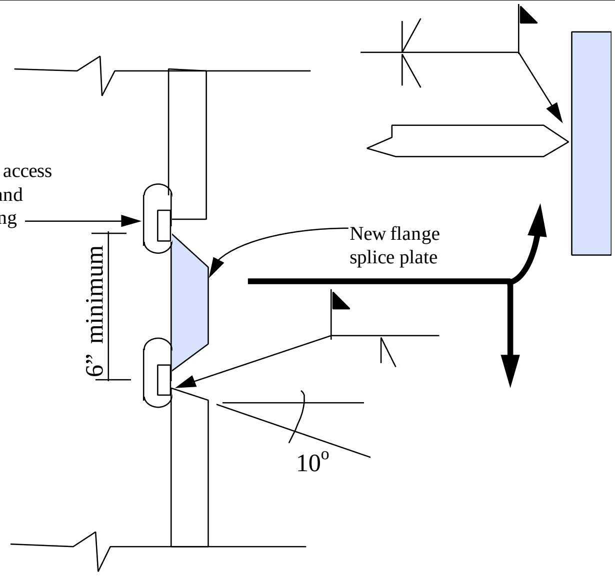
Table 6-2 - Reference Details for Type C and P Damage

Damage Class	Figure
Beam Access	Figure 6-4
C1	Figure 6-4, 6-5
C2	Figure 6-4, 6-6
C3	Figure 6-4, 6-5
C4	Figure 6-4, 6-5
C5	Figure 6-4, 6-6
P1	remove, prepare, replace
P2	arc-gouge and reweld
P4	arc-gouge and reweld
P5	Figure 6-7
P6	Figure 6-7
P7	Figure 6-7
P8	Figure 6-8



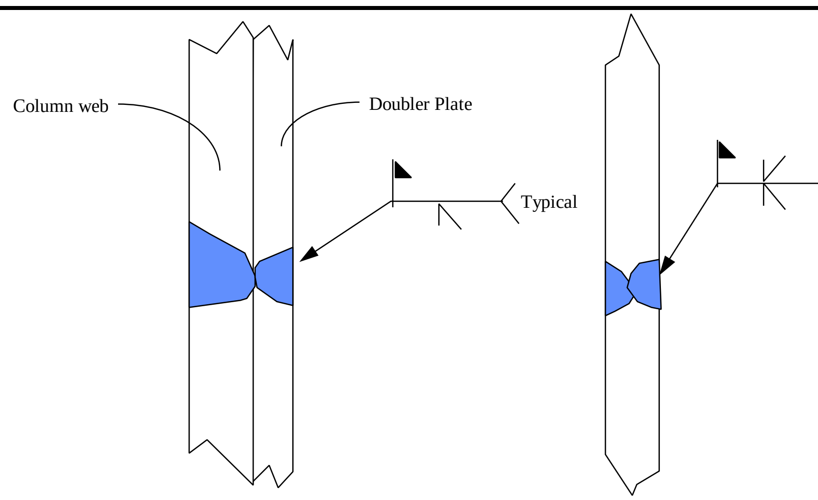
1 Investigate extent of fracture by UT to confirm that fractures are contained with the 45 degree angle zone of a standard pre-qualified CJP groove weld as defined by AWS D1.1, Figure 2.4, Joint Designation B-U4a-G
2 Provide 10° bevel on lower flange plate, to channel slag out of joint.
3 Grind all surfaces upon which weld metal will be deposited to smooth, uniform surface.

Figure 6-5 - Backgouge and reweld of column flange



Note: Provide new flange plate material of the same strength, and width as the existing column flange. Align rolling direction of plate with that of column flange. New plate should be of the same thickness as the existing flange with a tolerance of -0"/+1/4." The welding should be sequenced to connect the column flange to new flange plate welds prior to welding the column web to new flange plate. Bevel the lower edge of the column flange, and upper edge of the splice plate down 10°, to channel slag out of joint.

Figure 6-6 - Replacement of Column Flange Repair

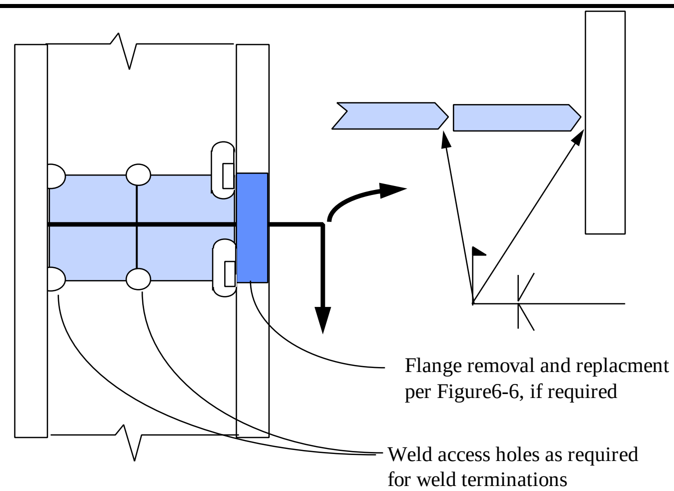


Web with Doubler Plate Web without Doubler Plate

Notes:

1. Prepare fractured section of doubler by air-arc gouging, grind and reweld, using web as backing
2. Prepare fractured section of web by air-arc gouging, grind and reweld, using doubler as backing or backgouge and reweld from reverse side, if no doubler present.

Figure 6-7 - Reweld Repair of Web plate and Doubler plate



Notes:

1. Sequence removal of portions of column and provide shoring as required to safely support existing column loads.
2. Thickness of new web plate to match existing column web (tolerance +1/8”, -0”).

Figure 6-8 - Alternate Column Web Repair - Columns without Doubler Plates

Commentary: Special attention should be given to conditions where more than 20% of the column cross section will be removed at one time, as special temporary shoring may be warranted. In addition, care should be taken when applying heat to a flange or web containing a fracture, as fractures have been observed to propagate with the application of heat. This can be prevented by drilling a small diameter hole at the end of the fracture, to prevent it from running.

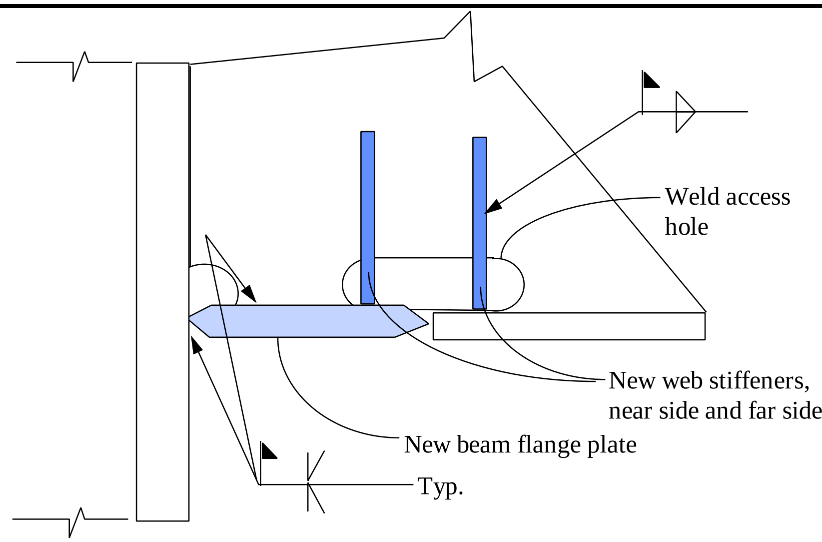
6.3.4 Column splice fractures - Type C7

Any fractures detected in column splices should be repaired by removing the fractured material and replacing it with sound weld material. For partial joint penetration groove welds, remove up to one half of the material thickness from one side and replace with sound material. Where complete joint penetration groove welds are required, it may be preferable to provide a double bevel weld, repairing one half of the material thickness completely prior to preparing and repairing the other half. Alternatively, if calculations indicate that column loads may safely be resisted with the entire section of column flange removed, or if suitable shoring is provided, it may be preferable to use a single bevel weld.

Commentary: Special attention should be given to these conditions, as the removal of material may require special temporary shoring. Also, since partial penetration groove welds can serve as fracture initiators in tension applications, *consideration should be given to replacing such damaged splice areas with complete joint penetration welds.*

6.3.5 Girder Flange Fractures - Type G3-G5

Repair of fractures in girder flanges may be performed by several methods. One method is to remove the fracture by air arc gouging such that no sign of discontinuity or defect within a six (6) inch radius around the fracture remains, preparing the surface by grinding and welding new material back. Alternatively, damaged portions of the girder flange may be removed and replaced with new plate as shown in Figure 6-9 or Figure 6-10.



Notes:

1. New plate thickness to match beam flange thickness + height of removed web fillet.
2. Weld sequence - a) weld of new flange plate to column; b)weld of flange plate stiffeners to web and flange plate; c) weld of new flange plate to beam flange. d)weld of stiffener plate to beam flange and web

Figure 6-9 - Beam Flange Plate Replacement

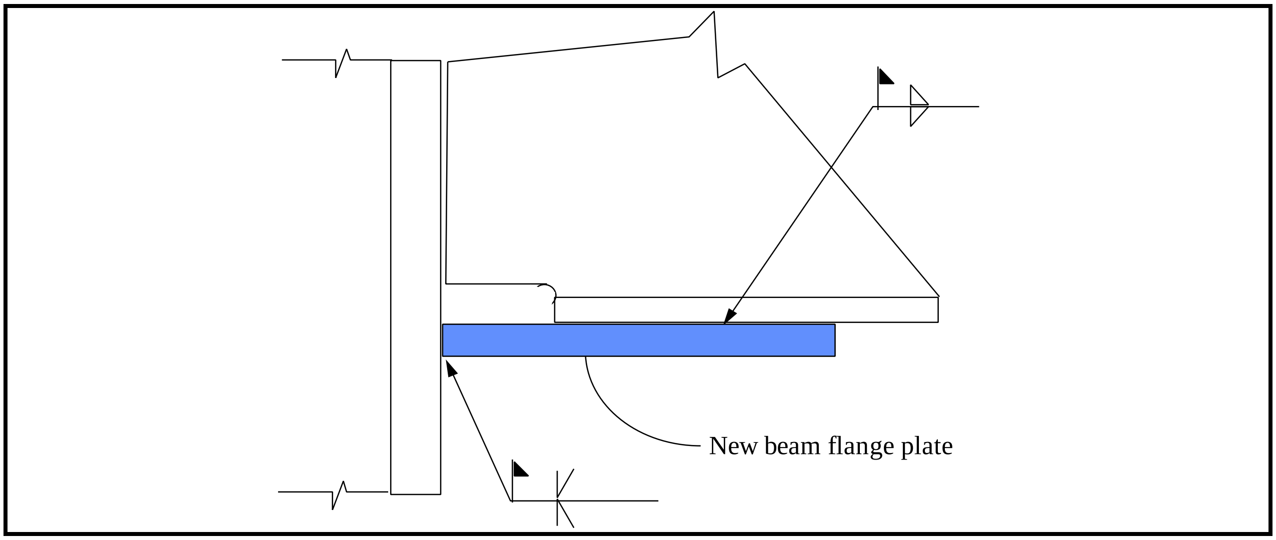


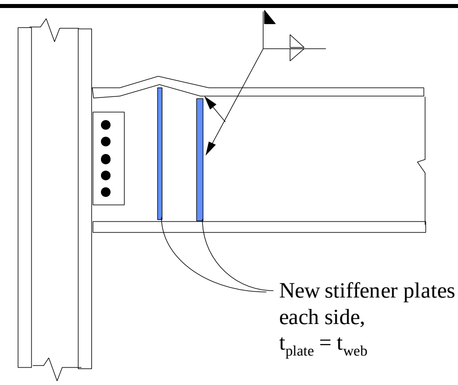
Figure 6-10 - Alternative Beam Flange Plate Replacement

Commentary: Due to accessibility difficulties or excessive weld build-up requirements, it may become necessary to remove a portion of the girder flange to properly complete the joint repair. A minimum of six inches of girder flange may be removed to facilitate the joint repair, with the optimum length being equal to the flange width. After removal of the portion of flange, the face of column and cut edge of girder flange may then be prepared to receive a splice plate matching the flange in grade and width. Thickness should be adjusted as required to make-up the depth of the girder web and fillet removed as part of the preparation process.

It is recommended that a double bevel joint be utilized in replacing the removed plate to eliminate the need for backup bars, consequently also eliminating the removal of these backup bars. A suggested joint detail is a B-U3/TC-U5, per AWS D1.1, with 1/3 t_{flange}-2/3 t_{flange} bevels on the plate. The web of the girder should be prepared at the column and butt weld areas to allow welding access. Weld tabs may be used at the column and butt weld. The weld between the splice plate and the column flange should be completed first. If a double bevel weld is selected, the welder may choose to weld the first few passes from one face, then backgouge and weld from the second side. This may help to keep the interpass temperature below the maximum without down time often encountered in waiting for the weld to cool.

6.3.6 Buckled Girder Flanges - Type G1

Where the top or bottom flange of a girder has buckled, and the rotation between the flange and web is less than or equal to the mill rolling tolerance given in the AISC Manual of Steel Construction (AISC-1994 or AISC-1989) the flange need not be repaired. Where the angle is greater than mill rolling tolerance, repair should be performed and may consist of adding full height stiffener plates on the web over each portion of buckled flange, contacting the flange at the center of the buckle, (Figure 6-11) or using heat straightening procedures. Another available approach is to remove the buckled portion of flange and replace it with plate, similar to Figures 6-9 and 6-10.



Note: Provide stiffeners at beginning of buckle and at center of buckle

Figure 6-11 - Addition of Stiffeners at Buckled Girder Flange

Commentary: Should flange buckling occur on only one side of the web, and the buckle repair consists of adding stiffener plates, only the side that has buckled need be stiffened. In case of partial flange replacement, special shoring requirements should be considered by the design engineer.

6.3.7 Buckled column flanges - Type C6

Any column flange or portion of a flange that has buckled to the point where it exceeds the rolling tolerances given in the AISC Manual of Steel Construction should be repaired. Flange repair may consist either of flame straightening or of removing the entire buckled portion of flange and replacing it with material with yield properties similar to the actual yield properties of the damaged material similar to Figure 6-6. If workers with the appropriate skill to perform flame straightening are available, this is the preferred method.

Commentary: For flange replacement, shoring is normally required. This shoring should be designed by the structural engineer, or may be designed by the contractor provided the design is reviewed by the structural engineer.

Flame straightening can be an extremely effective method of repairing buckled members. It is performed by applying heat to the member in a triangular pattern, in order to induce thermal strains that straighten the member out. Very large bends can be straightened by this technique. However, the practice of this technique is not routine and there are no standard specifications available for controlling the work. Consequently, the success of the technique is dependent on the availability of workers who have the appropriate training and experience to perform the work. During the heat application process, the damaged member is locally heated to very high temperatures. Consequently, shoring may be required for members being straightened in this manner.

A number of references are available that provide more information on this process and its applications, published by AISC and others (Avent - 1992), (Avent - 1995), (Shonafelt and Horn - 1984)

6.3.8 Gravity connections

Connections not part of the lateral load-resisting system may also be found to require repair due to excessive rotation or demand caused by distress of the lateral load-resisting system in the zone of influence. These connections should be repaired to a capacity at least equivalent to the pre-damaged connection capacity. Shear connections that are part of the lateral load resisting system should be repaired in a similar manner, with special consideration given to the nature and significance of the overall structural damage. In buildings which are repaired, but not modified, future earthquakes may cause moment connection failures with resulting large building deflections and high rotation demands at gravity connections. When repairing gravity connections, consideration should be given to providing connections with the ability to rotate with little or no reduction in vertical load carrying capacity, possibly by dissipating energy (through the use of slip critical bolts with horizontal short slotted holes).

Commentary: In many cases, shear connections which were not a part of the lateral-force-resisting system provided an unanticipated redundancy after damage occurred to the primary WSMF lateral system. While repair details could provide for rotation to minimize damage, such details should not eliminate the beneficial effect of the extra strength and stiffness these shear connections provide. This is especially important in framing systems with low moment frame redundancy.

The suggestion of providing gravity connections with slotted holes and slip critical bolts may be a reasonable compromise. Such a connection would be capable of providing some additional, unintended, strength and stiffness for the building but would also be able to withstand relatively large rotations without jeopardizing the gravity support the connection is actually intended to provide.

6.3.9 Reuse of Bolts

Bolts in a connection displaying bolt damage or plate slippage should not be re-used. As indicated in the AISC Specification for Structural Joints using ASTM A325 or A490 Bolts (American Institute of Steel Construction - 1985), A490 bolts and galvanized A325 bolts should not be retightened and re-used under any circumstances. Other A325 bolts may be reused if determined to be in good condition. Touching up or retightening previously tightened bolts which may have been loosened by the tightening of adjacent bolts need not be considered as reuse provided the snugging up continues from the initial position and does not require greater rotation, including the tolerance, than that required by Table 5 of the AISC Specification. Bolts in connections displaying bolt or plate slippage should not be reused.

Commentary: Proper performance of high strength bolts used in slip critical applications requires proper tensioning of the bolt. Although a number of *methods are available to ensure that bolts are correctly tensioned, the most common methods relate to torquing of the nut on the bolt. When a bolt has been damaged, the torquing characteristics will be altered. As a result, damaged bolts may either be over-tightened or under-tightened, if reinstalled. The threads of ASTM A-490 bolts and galvanized ASTM A-325 bolts become slightly damaged when tightened, and consequently, should not be reused. To determine if an ungalvanized ASTM A-325 bolt is suitable for re-use, a nut should be run up the threads of the bolt. If this can be done smoothly, without binding, then the bolt may be re-used.*

6.3.10 Welding Specification

Welded repairs involving thick plates and conditions of high restraint should be specified with caution. These conditions can lead to large residual stresses and in some cases, initiation of cracking before the structure is loaded. The potential for problems can be reduced by specifying appropriate joint configurations, welding processes, control of preheat, heat input during welding and cooldown, as well as selecting electrodes appropriate to the application. Engineers who do not have adequate knowledge to confidently specify these parameters should seek consultation from a person with the required expertise.

6.4 Preparation

6.4.1 Welding Procedure Specifications

A separate Welding Procedure Specification (WPS) should be established for every different weld configuration, welding position, and material specification. Two categories of qualified welding procedures are given in AWS D1.1-94. The WPS should be reviewed by the structural engineer responsible for the repairs. The WPS is a set of focused instructions to the welders and inspectors stating how the welding is to be accomplished. Each type of weld should have its own WPS solely for the purpose of that weld. The WPS should include instructions for joint preparation based on material property and thickness, as well as welding parameters. Weld process, electrode type, diameter, stick-out, voltage, current, and interpass temperature should be clearly defined. In addition, joint preheat and postheat requirements should be specified as appropriate, including insulation guidelines if applicable. The WPS should also list appropriate interim specification requirements that are mandated by the project specification.

Commentary: Preparation of the WPS is normally the responsibility of the fabricator/erector. Sample formats for WPS preparation and submission are included in AWS D1.1. Some contractors fill out the WPS by inserting references to the various AWS D1.1 tables rather than the actual data. This does not meet the intent of the WPS which is to provide specific instructions to the welder and inspector on how the weld is to be performed. The actual values of the parameters to be used should be included in the WPS submittal.

6.4.2 Welder Training

Training of welders should take place at the outset of the repair operations. Welders and inspectors should be familiar with the WPS, and should be capable of demonstrating familiarity with each of its aspects. A copy of the WPS should be located on site, preferably at the connection under repair, accessible to all parties involved in the repair.

6.4.3 Welder Qualifications

Welders must be qualified and capable of successfully making the repair welds required. All welders should be qualified to the AWS D1.1 requirements for the particular welding process and position in which the welding is to be performed. Successful qualification to these requirements, however, does not automatically demonstrate a welder's ability to make repair welds for all the configurations that may be encountered. Specific additional training and/or experience may be required for repair situations. Welders performing repairs should have a minimum of two years of verifiable field experience for the welding process that is employed, as well as experience in arc-gouging and thermal cutting of material. Inexperienced welders should demonstrate their ability to make proper repair welds. This may be done by welding on a mock-up assembly (see Section 6.4.4) that duplicates the types of conditions that would be encountered on the actual project. Alternatively, the welder could demonstrate proficient performance on the actual project, providing this performance is continuously monitored, start to finish, during the construction of at least the first weld repair. This observation should be made by a qualified welding inspector or Welding Engineer.

6.4.4 Joint Mock-ups

A joint mock-up should be considered as a training and qualification tool for each type of repair the welder is to perform that is more challenging than work in which he/she has previously demonstrated competence, or at the discretion of the structural engineer. This will allow the welder to become familiar with atypical welds, and will give the inspector the opportunity to clearly observe the performance of each welder. An entire mock-up is recommended for each such case, rather than only a single pass or portion of the weld as all welding positions and types of weld would be experienced, thus showing the welder capable of successfully completing the weld in all required positions, and applying all heating requirements.

6.4.5 Repair Sequence

Repair sequence should be considered in the design of repairs, and any sequencing requirements should be clearly indicated on the drawings and WPS. Structural instabilities or high residual stresses could arise from improper sequencing. The order of repair of flanges, shear plates, fractured columns, etc. should be indicated on the drawings to reduce possible residual stresses.

6.4.6 Concurrent Work

The maximum number of connections permitted to be repaired concurrently should be indicated on the drawings or in the project specifications.

Commentary: Although a connection is damaged, it may still posses significant ability to participate in the structure's lateral load resisting system. Consideration should be given to limiting the total number of connections being repaired at any one time, as the overall lateral load resistance of the structure may be temporarily reduced by some repair operations. If many connections are under repair simultaneously, the overall lateral resistance of the remaining frame connections may not be adequate to protect the structure's stability. Although this appears to fall under the category of means and methods, the typical contractor would have no way of determining the maximum number of connections that can be repaired at any one time without requiring supplemental lateral bracing of the building during construction. Therefore, the structural engineer should take a pro-active role in determining this.

6.4.7 Quality Control/Quality Assurance:

Quality control and quality assurance should follow the guidelines set forth in Section 6.6 and Chapters 9, 10 and 11 of these Interim Guidelines.

6.5 Execution

6.5.1 Introduction

Recommended general requirements should include the following:

1. Strict enforcement of the welding requirements in AWS D1.1 as modified in 1994 *UBC* Chapter 22, Division VIII or IX.

Commentary: Following the 1994 Northridge Earthquake, the AWS established a presidential task group to determine if deficiencies in the D1.1 code contributed to the unexpected damage, and to determine if modifications to the code should be made. That task group noted some areas of practice, related to steel moment frames in seismic zones, that could be improved relative to D1.1. These included the following recommendations:

a) the root pass of the complete joint penetration welds of beam to column flanges should not exceed 1/4 inch in size, for prequalified procedures.

b) where notch tough weld metal is desired, such as at the critical complete joint penetration welds of beam flanges to columns, the maximum interpass temperature should not exceed 550^{o}.

c) *when a FCAW process is used, the welding procedure specification should conform to the electrode manufacturer's recommendations.*

d) *the criteria for joints loaded in tension should apply to both top and bottom flange connections in moment frames.*

Future editions of the AWS D1.1 code may adopt some or all of these recommendations. In the interim period, the structural engineer should consider including these recommendations in the project welding specifications, to supplement the standard AWS D1.1 requirements.

2. Implementation of the special inspection requirements in *1994 UBC* section 1701 {*NEHRP-91* Section 1.6.2.6} and AWS D1.1. Visual inspection means that the inspector inspects the welding periodically for adherence to the approved Welding Procedure Specification (WPS) and AWS D1.1 starting with preliminary tack welding and fit-up and proceeding through the welding process. Reliance on the use of nondestructive testing (NDT) at the end of the welding process alone should be avoided. Use visual inspection in conjunction with NDT to improve the chances of achieving a sound weld.

3. Require the fabricator to prepare and submit a WPS with at least the information required by AWS D1.1 as discussed in Section 4.

4. Welding electrodes should be capable of depositing weld metal with a minimum notch toughness as described in Chapter 8.

5. All welds for the frame girder-column joints should be started and ended on weld run-off tabs where practical. All weld tabs should be removed, the affected area ground smooth and tested for defects using the magnetic particle method. Acceptance criteria should be AWS D1.1, section 8.15.1. Imperfections less than 1/16" should be removed by grinding. Deeper gouges, areas of lack of fusion, slag inclusions, etc., should be removed by gouging or grinding and rewelding following the procedures outlined above.

6. Weld dams do not meet the intent of weld tabs, are not permitted by AWS D1.1, and should not be permitted in the work. Dams are not necessary when proper bead size limitations are observed.

7. Steel backing (backing bars), if used, should be removed from new and/or repaired welds at the girder bottom flange, the weld root back-gouged by air arcing and the area tested for defects using the magnetic particle method, as described above. The weld should be completed and reinforced with a fillet weld. Removal of the weld backing at repairs of the top girder flange weld may be considered, at the discretion of the structural engineer.

Prior to removing weld backing, the contractor should prepare and submit a written WPS for review by the structural engineer. The WPS should conform to the requirements of AWS D1.1. In addition, a WPS should be prepared for each welding process to be used on the project and should include minimum preheat, maximum interpass temperatures, and the as-gouged cross section which must simulate a prequalified joint design of D1.1. If for any reason the WPS does not meet the prequalified limits of AWS it should be qualified by test, in accordance with Section 5.2 of AWS D1.1 In addition the contractor should propose the method(s) which will be used to remove the weld backing, back gouge to sound metal and when during this process he will apply preheat.

Although project conditions may vary, the following general guidelines may be considered:

> The steel backing may be removed by either grinding or by the use of air arc, or oxy-fuel gouging. The zone just beyond the theoretical 90 degree intersection of the beam to column flange should be removed by either air arc or oxy-fuel gouging followed by a thin grinding disk, or by a grinding disk alone. This shallow gouged depth of weld and base metal should then be tested by MT to determine if any linear indications remain. If the area is free of indications the area may then be re-welded. The preheat should be maintained and monitored throughout the process. If no further modification is to be made or if the modification will not be affected by a reinforcing fillet weld, the reinforcing fillet may be welded while the connection remains at or above the minimum preheat temperature and below the maximum interpass temperature.

If weld tabs were used and are to be removed in conjunction with the removal of the weld backing, the tabs should be removed after the weld backing has been removed and fillet added. If cover plates are to be added, the removal of the weld tabs may occur before or after the plate is added depending on the width and configuration of the plate. This sequence should be submitted to the structural engineer for his/her approval prior to the beginning of the work.

The weld tabs may be removed by air arc or oxy-fuel gouging followed by grinding or by grinding alone. The resulting contour should blend smoothly with the face of the column flange and the edge of the beam flange and should have a radius of 1/4-3/8 inch.

The finished surface should be visually inspected for contour and any visually apparent indications. This should be followed by magnetic particle testing (MT). Linear indications found in this location of the weld may be detrimental. They may be the result of the final residue of defects commonly found in the weld tab area. Linear indications should be removed by lightly grinding or using a cutting tool until the indication is removed. If after removal of the defect the ground area can be tapered and is not beyond the theoretical 90 degree intersection of the beam flange edge and column flange, weld repair may not be necessary and should be avoided if possible.

If the defect removal has extended into the theoretical weld section, then weld repair may be necessary. The weld repair should be performed in accordance with the contractor’s WPS, with strict adherence to the preheat requirements.

The surface should receive a final visual inspection and MT after all repairs and surface conditioning has been completed.

End dams, if present, should be removed if UT indicates rejectable flaws in the area of the end dam. Prior to removal of end dams, the contractor should submit a removal / repair plan which lists the method of dam removal, defect removal, welding procedure including, process, preheat, and joint configuration. The tab may be removed by grinding, air arc or oxy-fuel torch.

Any weld defects should be removed by grinding or cutting tools, or by air arc gouging followed by grinding. The individual performing defect removal should be furnished the UT results which describe the location depth and extent of the defect(s).

When the individual removing the defects has completed this operation, and has visually confirmed that no remnants remain, the surface should be tested by MT. Additional defect removal and MT may occur until the MT tests reveal that the defects have been removed.

The contour of the surface at this point may be too irregular in profile to allow welding to begin. The surface should be conditioned by grinding or using a cutting tool to develop a joint profile which conforms to the WPS. Prior to welding MT should be performed to determine if any additional defects have been exposed.

Based upon a satisfactory MT the joint may be prepared for welding. Weld tabs (and backing if necessary) should be added. The welding may begin and proceed in accordance with the WPS. The theoretical weld must be completed for its full height and length. Careful attention should be paid to ensure that weld bead size does not exceed that permitted by the WPS.

If specified, the weld tabs and backing should be removed in accordance with the guideline section describing this technique. The final weld should be inspected by MT and UT.

Commentary: Removal of the weld backing from the top flange may be difficult, particularly along perimeter frames where access to the outer side is restricted. Since the potential stress riser produced by the unwelded portions of the weld backing are not located on the extreme outer fiber of the frame girder, the benefits of removal may be limited in repair situations. Nevertheless, there may be benefits to providing a weld with a more favorable contour (i.e. that produced by the reinforcing fillet). Tests conducted to date have not been conclusive with regard to the benefit of top flange weld backing removal. At this time, there is no direct evidence that removal of weld backing from continuity plates in the column panel zone is required.

The decision to remove end dams should be based upon the results of UT. Since numerous stop - starts have occurred in this section of the theoretical weld, rejectable edge indications may reduce the integrity of the weld, especially during dynamic or seismic loading. If, however the area is found acceptable by UT removal is not necessary.

Excessive weaving of the weld bead, which can lead to unacceptable stresses at the toe of each weave, should not be allowed. However, some oscillation of the electrode may be required to obtain good fusion.

6.5.2 Girder Repair

If at bottom flange repairs back gouging removes sufficient material such that a weld backing is required for the repair, after welding the backing should be removed from the girder. Alternatively, a double-beveled joint may be used The weld root should be inspected and tested for imperfections, which if found, should be removed by back-gouging to sound material. A reinforcing fillet weld should be placed at "T" joints equal to one-quarter of the girder flange thickness. It need not exceed 3/8 inch (see Note J, Figure 2.4 of AWS D1.1.)

If the bottom flange weld requires repair, the following procedure may be considered:

1. The root pass should not exceed a 1/4 inch bead size.

2. The first half-length root pass should be made with one of the following techniques, at the option of the contractor:

    a) The root pass may be initiated near the center of the joint. If this approach is used, the welder should extend the electrode through the weld access hole, approximately 1" beyond the opposite side of the girder web. This is to allow adequate access for clearing and inspection of the initiation point of the weld before the second half-length of the root pass is applied. It is not desirable to initiate the arc in the exact center of the girder width since this will limit access to the start of the weld during post-weld operations. After the arc is initiated, travel should progress towards the end of the joint (outboard beam flange edge), and the weld should be terminated on a weld tab.

    b) The weld may be initiated on the weld tab, with travel progressing toward the center of the girder flange width. When this approach is used, the welder should stop the weld approximately 1" before the beam web. It is not advisable to leave the weld crater directly in the center of the beam flange width since this will hinder post-weld operations.

3. The half length root pass should be thoroughly slagged and cleaned.

4. The end of the half length root pass that is near the center of the beam flange should be visually inspected to ensure fusion, soundness, freedom from slag inclusions and excessive porosity. The resulting bead profile should be suitable for obtaining fusion by the subsequent pass to be initiated on the opposite side of the girder web. If the profile is not conducive to good fusion, the start of the first root pass should be ground, gouged, chipped or otherwise prepared to ensure adequate fusion.

5. The second half of the weld joint should have the root pass applied before any other weld passes are performed. The arc should be initiated at the end of the half length root pass that is near the center of the beam flange, and travel should progress to the outboard end of the joint, terminating on the weld tab.

6. Each weld layer should be completed on both sides of the joint before a new layer is deposited.

7. Weld tabs should be removed and ground flush to the beam flange. Imperfections less than 1/16" should be removed by grinding. Deeper gouges, areas of lack of fusion, slag inclusions, etc. should be removed by gouging or grinding and rewelding following the procedures outlined above.

6.5.3 Weld Repair (Types W1, W2, or W3)

When W1, W2, or W3 cracks are found, the column base metal should be evaluated using UT to determine if fractures have progressed into the flange. This testing should be performed both during the period of discovery and during repair.

When a linear planar-type defect such as a crack or lack of fusion can be determined to extend beyond one-half the thickness of the beam flange, it is generally preferred to use a double-sided weld for repair (even though the fracture may not extend all the way to the opposite surface.) This is because the net volume of material that needs to be removed and restored is generally less when a double-sided joint is utilized. It also results in a better distribution of residual stresses since they are roughly balanced on either side of the center of the flange thickness.

Repair of these cracks may warrant total removal of the original weld, particularly if multiple cracks are present. If the entire weld plus some base metal is removed care must be taken not to exceed the root opening and bevel limits of AWS D1.1 unless a qualified by test WPS is used. If this cannot be avoided one of two options is available:

1. The beveled face of the beam and/or the column face may be built up (buttered) until the desired root opening and angle is obtained.

2. A section of the flange may be removed and a splice plate inserted.

Commentary: Building up base metal with welding is a less intrusive technique than removing large sections of the base metal and replacing with new plate. However, this technique should not be used if the length of build-up exceeds the thickness of the plate.

6.5.4 Column Flange Repairs - Type C2

Damage type C2 is a pullout type failure of the column flange material. The zone should be conditioned to a concave surface by grinding and inspected for soundness using MT. The concave area may then be built up by welding. The joint contour described in the WPS should specify a "boat shaped" section with a "U" shaped cross section and tapered ends. The weld passes should be horizontal stringers placed in accordance with the WPS. Since stop/starts will occur in the finished weld, care must be taken to condition each stop/start to remove discontinuities and provide an adequate contour for subsequent passes. The final surface should be ground smooth and flush with the column face. This surface and immediate surrounding area should be subjected to MT and UT.

6.6 STRUCTURAL MODIFICATION

6.6.1 Definition of Modification

Within the context of damage to WSMF connections, the term "structural modification" refers to alteration of the connection to improve its earthquake performance and that of the structure as a whole. This typically involves substantial changes to the connection's geometry, capacity, or relevant limit states (e.g. flexural or shear strength or stiffness). Work that includes removal of existing welds and replacement with welds of improved toughness and/or workmanship is not considered modification under these Interim Guidelines.

Commentary: This term is contrasted with "repair," wherein the essential behavior of the connection is unchanged as a result of the repair effort. Geometrical or stiffness changes can involve spatial alterations to the elements of the connection, such as adding column stiffeners or the addition of new connection elements, such as cover plates, upstanding ribs, side plates or haunches. Changes to the connection's capacity, either in flexure or shear, may occur as a result of the addition of new connection components. Altering the connection's relevant limit states may occur, for example, when the location of the plastic hinge is shifted away from its original location or the shear capacity of the connection or one of its elements determines the behavior of the connection.

Much of the damage that occurred in the Northridge Earthquake has been attributed to the presence of “crack like” conditions at the root of the complete joint penetration beam flange to column flange welds. These crack like conditions included lack of fusion at the weld root as well as the presence of partially fused weld backing. Some engineers believe that if these crack-like conditions are removed, substantial improvement in connection performance can be obtained. SAC conducted specific testing in the Phase 1 program in which such “dressing up” of these welds was performed. The performance of the connection in these tests was mixed, and often not substantially improved relative to that of connections in which the backing was left in place. Based on these tests, removal of weld backing, backgouging and repairing welds, and reinforcing with a fillet is not recommended as a means of connection modification, although it is an acceptable means of repair for joints with type W1 and W2 damage.

Several engineers and researchers knowledgeable in fracture mechanics have suggested that the standard, unreinforced moment connection could perform acceptably if weld metal and base metal with adequate toughness were incorporated, and beam flange to column joints are executed in such a manner that large crack-like discontinuities are not present (removal of backing and weld tabs, backgouging, and reinforcing with a fillet). Other engineers knowledgeable in mechanics of materials (Blodgett - 1994) believe that regardless of the toughness of the weld metal employed, the connection configuration is such that reliable performance is unlikely.

If joints with adequate weld metal toughness can provide substantially more reliable performance, then, removal of existing low-toughness welds and replacement with new tough material may be an acceptable means of modification. To date, only limited testing of such assemblies have been conducted. In one test (Popov - 1995) an assembly consisting of a W36 x 150 beam connected to a W14 x 257 column and originally fabricated using E70T4 electrodes (not having rated notch toughness) was repaired following initial testing by completely removing the complete joint penetration welds of the beam flanges to column flanges and replacing them with new welds made with electrodes having specified notch toughness. Weld backing and weld tabs were removed and the welds were reinforced with a fillet. The specimen was successfully tested to a plastic rotation of 0.04 radians. However, until additional research can be performed to quantify the reliability obtained through the use of notch tough weld metal, this is not recommended by itself as a method of modification in these Interim Guidelines.

Modification of the structure as a whole, as opposed to individual connection modifications, can be an effective means of obtaining more reliable performance. The addition of braced frames, shear walls, energy dissipation systems, base isolation, etc., can be used to reduce the total deformation demand induced in the structure by earthquakes, and consequently the need for the moment-resisting connections to resist large plastic rotation demands. Interim Guidelines for these types of modifications are not directly included in this document. However, sections on connection qualification presented below provide information that can be used to determine the plastic rotation capacity of existing connections in the building. Once this is determined, the effectiveness of proposed global modification measures can be assessed, as part of the design process.

6.6.2 Damaged vs. Undamaged Connections

Engineers should inform building owners that substantial improvements in the reliability of future earthquake performance of a WSMF building can be obtained by structural modification. Modification can be made at connections that have sustained damage as well as those that are undamaged. On the basis of cost, some owners may elect to modify those connections which have been damaged, and which will be repaired, but not other, undamaged connections. If a building has had only a few scattered connections damaged, such an approach will not result in any significant improvement in future building performance, and is not recommended. If a substantial number of connections in a building have been damaged and will be repaired, modification of these damaged connections may improve future building performance, depending on the distribution of damaged connections, throughout the building. Therefore, consideration of such an approach has been recommended in Chapter 4 of these Interim Guidelines.

If possible, it is recommended that the modification of connections follow a rational spatial distribution, so a to distribute the enhanced energy dissipation capacity (and ductility) throughout the building. As a minimum, structural modification should consider the effect of those modifications on the performance of the lateral system as well as on the performance of individual components of the frames. An appropriate analysis should be performed of the building, considering the modifications, to ensure that undesirable stiffness irregularities are not introduced or made more severe, and that excessive demand is not concentrated in connections unable to resist the applied loads or deformations. The effects of connection modifications on inelastic demands in adjacent columns and panel zones should be considered.

Commentary: Structural modification of connections will normally be performed as a means of enhancing the expected performance of the building in future earthquakes, by minimizing the potential for fractures. The intent of modification is to make the connection sufficiently strong that inelastic behavior of the frame will be controlled by the formation of plastic hinges within the girder spans.

Evaluation of statistical data on the types and distribution of damage experienced by 89 buildings affected by the Northridge Earthquake (Bonowitz & Youssef - 1995) indicates that the spatial distribution of damage other than small root indications (Type W1) has modest correlation with the distribution of high seismic demands predicted by traditional analytical approaches. The distribution of type W1 indications appear to be random. A modification scheme that selects connections on the basis of existing damage could therefore result in a random distribution of connections with improved performance characteristics. In such an approach, connections that may undergo high plastic rotation demands or may be part of a lateral system with limited redundancy might not be modified in favor of connections damaged as a result of poor workmanship. The result of this could be a modified system with only marginally improved behavior. Connections that have not been modified can be expected to have a significant failure rate in subsequent earthquakes, at near-elastic demand levels. Therefore, the amount of improvement obtained by modifying only the damaged connections is not directly quantifiable. Generally, as more connections in the building are modified, the potential performance of the building should improve.

An alternative approach, and one that appears to represent a more reliable method of ensuring that the earthquake performance of the lateral system is equivalent to that assumed at the time the WSMF was designed, is to modify all of *the connections. Tests on girder-column connections similar to those found in many buildings suggest that the traditional welded flange/bolted web connection cannot develop the rotational demands implicit in building code designs. Modified connections appear to represent one approach to achieve the required level of deformation capacity.*

Modification of only selected connections may be a cost-effective approach if the analysis can accurately predict the demand on the connections as well as the consequences of future connection failures in the modified and unmodified connections. The structural engineer should inform the building owner of the assumed benefits as well as the potential disadvantages of a scheme that modifies only a selected number of the connections. The reliability of analyses used to justify such a partial modification scheme is sensitive to the modeling assumptions and the ground motion input.

6.6.3 Criteria

Connection modification intended to permit inelastic frame behavior should be proportioned so that the required plastic deformation of the frame may be accommodated through the development of plastic hinges at pre-determined locations within the girder spans, as indicated in Figure 6-12. Beam-column connections should be designed with sufficient strength (through the use of cover plates, haunches, side plates, etc.) to force development of the plastic hinge away from the column face. This condition may also be attained through local weakening of the beam section, at the desired location for plastic hinge formation. All elements of the connection should have adequate strength to develop the forces resulting from the formation of the plastic hinge at the predetermined location, together with forces resulting from gravity loads.

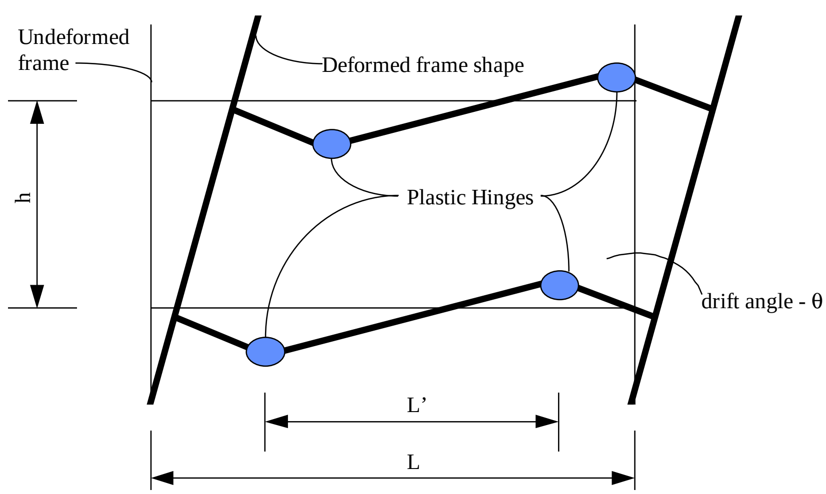


Figure 6-12 - Desired Plastic Frame Behavior

Commentary: Nonlinear deformation of frame structures is accommodated through the development of inelastic flexural or shear strains within discrete regions of the structure. At large inelastic strains these regions can develop into *plastic hinges, which can accommodate significant concentrated rotations at constant (or nearly constant) load through yielding at tensile fibers and buckling at compressive fibers. If a sufficient number of plastic hinges develop in a frame, a mechanism is formed and the frame can deform laterally in a plastic manner. This behavior is accompanied by significant energy dissipation, particularly if a number of members are involved in the plastic behavior, as well as substantial local damage to the highly strained elements. The formation of hinges in columns, as opposed to beams, is undesirable, as this results in the formation of mechanisms with relatively few elements participating, and consequently little energy dissipation occurring. In addition, such mechanisms also result in local damage to critical gravity load bearing elements.*

The prescriptive connection contained in the UBC and NEHRP Recommended Provisions prior to the Northridge Earthquake was based on the development of plastic hinges within the beams at the face of the column, or within the column panel zone itself. If the plastic hinge develops in the column panel zone, the resulting column deformation results in very large secondary stresses on the beam flange to column flange joint, a condition which can contribute to brittle failure. If the plastic hinge forms in the beam, at the face of the column, this can result in very large through-thickness strain demands on the column flange material and large inelastic strain demands on the weld metal and surrounding heat affected zones. These conditions can also lead to brittle joint failure. In order to achieve more reliable performance, it is recommended that the connection of the beam to the column be modified to be sufficiently strong to force the inelastic action (plastic hinge) away from the column face. Plastic hinges in steel beams have finite length, typically on the order of half the beam depth. Therefore, the location for the plastic hinge should be shifted at least that distance away from the face of the column. When this is done, the flexural demands on the columns are increased. Care must be taken to assure that weak column conditions are not inadvertently created by local strengthening of the connections.

It should be noted that connection modifications of the type described above, while believed to be effective in preventing brittle connection fractures, will not prevent structural damage from occurring. Brittle connection fractures are undesirable because they result in a substantial reduction in the lateral-force-resisting strength of the structure which, in extreme cases, can result in instability and collapse. Connections modified as described in these Interim Guidelines should experience many fewer such brittle fractures than unmodified connections. However, the formation of a plastic hinge within the span of a beam is not a completely benign event. Beams which have formed such hinges may exhibit large buckling and yielding deformation, damage which typically must be repaired. The cost of such repairs could be comparable to the costs incurred in repairing fracture damage experienced in the Northridge Earthquake. The primary difference is that life safety protection will be significantly enhanced and *most structures that have experienced such plastic deformation damage should continue to be safe for occupancy, while repairs are made.*

If the types of damage described above are unacceptable for a given building, then alternative methods of structural modification should be considered, that will reduce the plastic deformation demands on the structure during a strong earthquake. Appropriate methods of achieving such goals include the installation of supplemental braced frames, energy dissipation systems, and similar systematic modifications of the building's basic lateral force resisting system.

6.6.4 Strength

When these Interim Guidelines require determination of the strength of a framing element or component, this shall be calculated in accordance with the criteria contained in *UBC-94*, Section 2211.4.2 {*NEHRP-91* Section 10.2, except that the factor ϕ should be taken as 1.0}, restated as follows:

2211.4.1 Member strength. Where this section requires that the strength of the member be developed, the following shall be used:

Flexure	$M_s = Z F_y$
Shear	$V_s = 0.55 F_y d t$
Axial compression	$P_{sc} = 1.7 F_a A$
Axial tension	$P_{st} = F_y A$
Connectors	
Full Penetration welds	$F_y A$
Partial Penetration welds	1.7 allowable (see commentary)
Bolts and fillet welds	1.7 allowable

Commentary: Partial penetration welds are not recommended for tension applications in critical connections resisting seismic induced stresses. The geometry of partial penetration welds creates a notch-like condition that can initiate brittle fracture under conditions of high tensile strain.

6.6.5 Plastic Rotation Capacity

The plastic rotation capacity of modified connections should reflect realistic estimates of the required level of plastic rotation demand. In the absence of detailed calculations of rotation demand, connections should be shown to be capable of developing a minimum plastic rotation capacity on the order of 0.025 to 0.030 radian. The demand may be lower when braced frames, supplemental damping, base isolation, or other elements are introduced into the moment frame system, to control its lateral deformation; when the design ground motion is relatively low in the range of predominant periods for the structure; and when the frame is sufficiently strong.

If calculations are performed to determine the required connection plastic rotation capacity, the capacity should be taken somewhat greater than the calculated deformation demand, due to the high variability and uncertainty inherent in predictions of inelastic seismic response. Until better guidelines become available, a required plastic rotation capacity on the order of 0.005 radians greater than the demand calculated for the design basis earthquake (or if greater conservatism is desired - the maximum capable earthquake) is recommended. Rotation demand calculations should consider the effect of plastic hinge location within the beam span, as indicated in Figure 6-12, on plastic rotation demand. Calculations should be performed to the same level of detail specified for nonlinear dynamic analysis for base isolated structures in *UBC-94* Section 1655 {*NEHRP-94* Section 2.6.4.4}. Ground motion time histories utilized for these nonlinear analyses should satisfy the scaling requirements of *UBC-94* Section 1655.4.2 {*NEHRP-94* Section 2.6.4.4} except that instead of the base isolated period, T_I, the structure period, T, calculated in accordance with *UBC-94* Section 1628 {*NEHRP-94* Section 2.3.3.1} should be used.

Commentary. Traditionally, structural engineers have calculated demand in moment frames by sizing the members for strength and drift using code forces (either equivalent static or reduced dynamic forces) and then "developing the strength of the members." Since 1988, "developing the strength" has been accomplished by prescriptive means. It was assumed that the prescribed connections would be strong enough so that the girder would yield (in bending), or the panel zone would yield (in shear) in a nearly perfectly plastic manner producing the plastic rotations necessary to dissipate the energy of the earthquake. It is now known that the prescriptive connection is often incapable of behaving in this manner.

In the 1994 Northridge earthquake, many moment-frame connections fractured with little evidence of plastic hinging of the girders or yielding of the column panel zones. Testing of moment frame connections both prior to and subsequent to the earthquake suggests that the standard welded flange-bolted web connection is unable to reliably provide plastic rotations beyond about 0.005 radian for all ranges of girder depths and often fails below that level. Thus, for frames designed for code forces and for the code drift limits, new connection configurations must be developed to reliably accommodate such rotation without brittle fracture.

In order to develop reasonable estimates of the plastic rotation demands on a frame's connections, it is necessary to perform inelastic time history analyses. For regular structures, approximations of the plastic rotation demands can be obtained from linear elastic analyses. Analytical research (Newmark and Hall - 1982) suggests that for structures having the dynamic characteristics of most WSMF buildings, and for the ground motions typical of western US earthquakes, the total frame deflections obtained from an unreduced (no R or R_w factor) dynamic analysis provide an approximate estimate of those which would be experienced by the inelastic structure. For the typical spectra contained in the building code, this would indicate expected drift ratios on the order of 1%. The drift demands in a real structure, responding inelastically tend to concentrate in a few stories, rather than being uniformly distributed throughout the structure's *height. Therefore, it is reasonable to expect typical drift demands in individual stories on the order of 1.5% to 2% of the story height. As a rough approximation, the drift demand may be equated to the joint rotation demand, yielding expected rotation demands on the order of perhaps 2%. Since there is considerable variation in ground motion intensity and spectra, as well as the inelastic response of buildings to these ground motions, conservatism in selection of an appropriate connection rotation demand is warranted.*

In recent testing of large scale subassemblies incorporating modified connection details, conducted by SAC and others, when the connection design was able to achieve a plastic rotation demand of 0.025 radians or more for several cycles, the ultimate failure of the subassembly generally did not occur in the connection, but rather in the members themselves. Therefore, the stated connection capacity criteria would appear to result in connections capable of providing reliable performance.

It should be noted that the connection assembly capacity criteria for the modification of existing buildings, recommended by these Interim Guidelines, is somewhat reduced compared to that recommended for new buildings (Chapter 7). This is typical of approaches normally taken for existing structures. For new buildings, these Interim Guidelines discourage building-specific calculation of required plastic rotation capacity for connections and instead, encourage the development of highly ductile connection designs. For existing buildings, such an approach may lead to modification designs that are excessively costly, as well as the modification of structures which do not require such modification. Consequently, an approach which permits the development of semi-ductile connection designs, with sufficient plastic rotation capacity to withstand the expected demands from a design earthquake is adopted. It should be understood that buildings modified to this reduced criteria will not have the same reliability as new buildings, designed in accordance with the recommendations of Chapter 7. The criteria of Chapter 7 could be applied to existing buildings, if superior reliability is desired.

When performing inelastic frame analysis, in order to determine the required connection plastic rotation capacity, it is important to accurately account for the locations at which the plastic hinges will occur. Simplified models, which represent the hinge as occurring at the face of the column, will underestimate the plastic rotation demand. This problem becomes more severe as the column spacing, L, becomes shorter and the distance between plastic hinges, L', a greater portion of the total beam span. In extreme cases, the girder will not form plastic hinges at all, but instead, will develop a shear yield, similar to an eccentric braced frame.

6.6.6 Connection Qualification and Design

Modified girder-column connections may be qualified by testing or designed using calculations. Qualification by testing is the preferred approach. Preliminary designs of connections to be qualified by test may be obtained using the calculation procedures of Section 6.6.6.3. The procedures of that section may also be used to calibrate previous tests of similar connection configurations to slightly different applications, by extrapolation. Extrapolation of test results should be limited to connections of elements having similar geometries and material specifications as the tested connections. Designs based on calculation alone should be subject to qualified independent third party review.

6.6.6.1 Qualification Test Protocol

Unless future testing programs reveal significant effects of dynamic loading rate or time history loading, a testing protocol similar to ATC-24, *Guidelines for Cyclic Seismic Testing of Components of Steel Structures* (Applied Technology Council - 1992), is recommended as the basis for qualification tests.

The testing program should replicate as closely as practical the anticipated conditions in the field, including such factors as:

a) Member sizes.

b) Material specifications.

c) Welding process, details and construction conditions.

d) Cover-plates, continuity plates, web tabs, bolts, and doubler plates.

e) Connection configuration (e.g., beams on both sides).

f) Induced stresses because of restraint conditions on the welds and connection members.

g) Axial load, where pertinent.

h) Gravity load, where significant.

The testing program should be organized to provide as much information as possible about the capability of the connections selected. The following minimum program is recommended:

a) Test two full size specimens of the largest representative beam/column assembly in the project.

b) Test one additional full size specimen for each beam/column assembly with significantly different interaction properties, such as beam flange width-thickness (b/t) ratio, panel zone stress/distortion, etc.

If any of the specimens fails to meet the qualification criteria, the connection should be redesigned and retested.

Where two-sided connections are used in the structure, and the type of connection being used can be expected to perform differently in a two-sided use than in one-sided use, it should be tested in the two-sided configuration as well as the one-sided. Two-sided connection assemblies can be expected to behave differently than one-sided assemblies, for example, when panel zone distortions will be significantly different, or when systems involve transfer of stress to the column by plates, welds, or other elements which are connected to the beams on both sides of the column.

Testing to include axial load should be considered when analysis indicates that significant tension can be expected to occur in a significant number of the columns represented by the specimen and where the connection type relies on the through-thickness strength of the column flanges. If the presence of a floor slab is anticipated to have significant influence on either the location or mechanism of the plastic hinge formed, than this should also be included in the test specimen.

Commentary: The use of connection configurations that have been qualified by test is the preferred approach. While the testing of all connection geometries and member combinations in any given building is not practical, the number of tests must be large enough to be meaningful yet small enough to not be unreasonably costly. Testing, within the limitations of test specimen simplification, has the advantage of being able to replicate fabrication and welding procedures, joint geometry and member size, and potential modes of failure. If the testing is done in a manner consistent with other testing programs, reasonable comparisons can be made. On the other hand, testing is expensive and it is difficult to realistically test the girder-column connection using actual restraint conditions and earthquake loading rates. Calculations offer an economical alternative to testing that can accommodate different girder and column sizes, altered connection geometries, and member properties. Nevertheless, recent testing on girder-column connections from WSMFs casts doubt on some fundamental assumptions upon which the calculations are based and therefore, they should be used with caution.

Since the level of confidence in connections developed strictly on the basis of calculations may not be as high as those based on tests, the use of testing is encouraged. Tests are, however, relatively expensive and a reasonable degree of flexibility in interpreting the results of limited testing programs must be acknowledged.

How much extrapolation should be accepted is a difficult decision. As additional testing is done, more information may be available on what constitutes "conservative" testing conditions, thereby allowing easier decisions relative to extrapolating tests to actual conditions which are likely to be less demanding than the tests. For example, it is hypothesized that connections of shallower, thinner flanged members are likely to be more reliable than similar connections consisting of deeper, thicker flanged members. Thus, it may be possible to test the *largest assemblages of similar details and extrapolate to the smaller member sizes -- at least in comparable member group families.*

6.6.6.2 Acceptance Criteria

The minimum acceptance criteria for connection qualification for specimens tested in accordance with these Interim Guidelines should be as follows:

a) The connection should develop beam plastic rotations as indicated in Section 6.6.5, for at least one complete cycle.

b) The connection should develop a minimum strength equal to 80% of the plastic strength of the girder, calculated using minimum specified yield strength F_y, throughout the loading history required to achieve the required plastic rotation capacity, as indicated in a), above.

Commentary: Many connection configurations will be able to withstand plastic rotations on the order of 0.025 radians or more, but will have sustained significant damage and degradation of stiffness and strength in achieving this deformation. The intent of the acceptance criteria presented in this Section is to assure that when connections experience the required plastic rotation demand, they will still have significant remaining ability to participate in the structure's lateral load resisting system.

6.6.6.3 Calculations

All connections designs should be based on test data and the use of connections based upon calculations only is not recommended. An approved program of variations on the tested proto-typical connections may use calculations to assist in extrapolation of results.

Calculations should be correlated to tested material properties for base metals and welds. The properties should be those corresponding to the axes of loading of the base metal and weld in the joints and to the welding processes and materials intended for use. The tested properties may be specific to the materials and processes to be used in the project, or based on a statistically-based testing program. Use of properties inferred from other testing programs must be done with appropriate care and, where such inferred properties are used, designs should reflect the uncertainty inherent in such an indirect approach.

Calculations should initiate with the selection of a connection configuration, such as one of those indicated in Section 6.6.7, that will permit the formation of a plastic hinge within the beam span, away from the face of the column, when the frame is subjected to gravity and lateral loads.

6.6.6.3.1 Material Strength Properties

In the absence of project specific material property information (for example, mill test reports), the values listed in Table 6-3 should be used to determine the strength of steel shape and plate for purposes of calculation. The permissible strength for weld metal should be taken in accordance with the building code.

Table 6-3 - Properties for Use in Connection Modification Design

Material	F_y (ksi)	$F_{y\,m}$ (ksi)	F_u (ksi)
A36 Beam	36	[1]	[1]
Dual Certified Beam			
Axial, Flexural	50		65 min.
Shape Group 1		55[2]	
Shape Group 2		58[2]	
Shape Group 3		57[2]	
Shape Group 4		54[2]	
Through-Thickness	-	-	Note 3
A572 Column/Beam			
Axial, Flexural	50		65 min.
Shape Group 1		58[2]	
Shape Group 2		58[2]	
Shape Group 3		57[2]	
Shape Group 4		57[2]	
Shape Group 5		55[2]	
Through-Thickness	-	-	Note 3

Notes:
1. See Commentary
2. Based on coupons from web. For thick flanges, the $F_{y\,flange}$ is approximately 0.95 $F_{y\,web}$.
3. See Commentary

Commentary: Table 6-3, Note 1 - The material properties for steel nominally designated on the construction documents as ASTM A36 can be highly variable and in recent years, steel meeting the specified requirements for both ASTM A36 and A572 has routinely been incorporated in projects calling for A36 steel. Consequently, unless project specific data is available to indicate the actual strength of material incorporated into the project, the properties for ASTM A572 steel should be assumed when ASTM A36 is indicated on the drawings, and the assumption of a higher yield stress results in a more severe design condition.

Table 6-3, Note 3 - The causes for through-thickness failures of column flanges (types C2, C4, and C5), observed both in buildings damaged by the Northridge Earthquake and in some test specimens, are not well understood. They are thought to be a function of the metallurgy and "purity" of the steel; conditions of loading including the presence of axial load and rate of loading application; conditions of tri-axial restraint; conditions of local hardening and embrittlement within the weld's heat affected zone; and by the relationship of the connection components as they may affect flange bending stresses and flange curvature induced by panel zone yielding. Given the many complex factors which can affect the through-thickness strength of the column flange, determination of a reliable basis upon which to set permissible design stresses will require significant research.

Interim Recommendation No. 2 (SEAOC-1995) included a value of 40 ksi, applied to the projected area of beam flange attachment, for the through-thickness strength to be used in calculations. This value was selected because it was consistent with the successful tests of cover plated assemblies conducted at the University of Texas at Austin (Engelhardt and Sabol - 1994). However, because of the probable influence of all the factors noted above, this value can only be considered to reflect the specific conditions of those tests and specimens.

Although reduced stresses at the column face produced acceptable results in the University of Texas tests, the key to that success was more likely the result of forcing the plastic hinge away from the column than reduction of the through-thickness stress by the cover plates. Reduction of through-thickness column flange stress to ever lower levels by the use of thicker cover plates is not recommended, since such cover plates will result in ever higher forces on the face of the column flange.

Notwithstanding all of the above, successful tests using cover plates and other measures of moving hinges (and coincidentally reducing through-thickness stress) continue to be performed. In the interim, structural engineers choosing to utilize connections relying on through-thickness strength should recognize that despite the successful testing, connections relying on through-thickness strength can not be considered to be fully reliable until the influence of the other parameters discussed above can be fully understood. A high amount of structural redundancy is recommended for frames employing connections which rely on through-thickness strength of the column flange.

6.6.6.3.2 Determine Plastic Hinge Location

The desired location for the formation of plastic hinges should be determined as a basic parameter for the calculations. For beams with gravity loads representing a small portion of the total flexural demand, the plastic hinge may be assumed to occur at a distance equal to 1/3 of the beam depth from the edge of the reinforced connection (or start of the weakened beam section), unless specific test data for the connection indicates that a different value is appropriate. Refer to Figure 6-13.

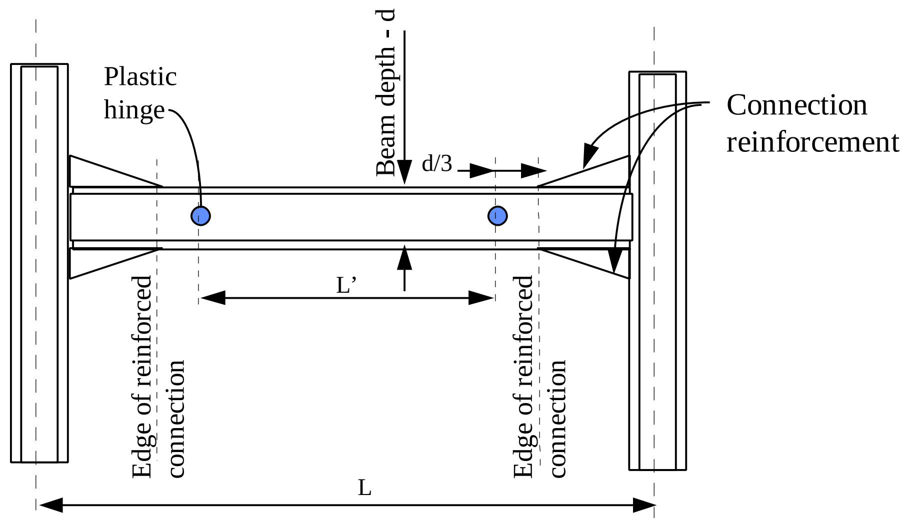


Figure 6-13 - Location of Plastic Hinge

Commentary: The suggested location for the plastic hinge, at a distance d/3 away from the end of the reinforced section is based on the observed behavior of test specimens, with no significant gravity load present. If the significant gravity load is present, this can shift the locations of the plastic hinges, and in the extreme case, even change the form of the collapse mechanism. If flexural demand on the girder due to gravity load is less than about 30% of the girder plastic capacity, this effect can safely be neglected, and the plastic hinge locations taken as indicated. If gravity demands significantly exceed this level then plastic analysis of the girder should be performed to determine the appropriate hinge locations. Note that in zones of high seismicity (UBC Zones 3 and 4, and NEHRP Map Areas 6 and 7) gravity loading on the girders of earthquake resisting frames typically has a very small effect.

6.6.6.3.3 Determine Probable Plastic Moment at Hinges

The probable value of the plastic moment, M_{pr}, at the location of the plastic hinges should be determined from the equation:

$$M_{pr} = 0.95\alpha Z_b F_{ya} \qquad (6\text{-}1)$$

where: α is a coefficient that accounts for the effects of strain hardening and modeling uncertainty, taken as:

1.1 when qualification testing is performed or calculations are correlated with previous qualification testing

1.3 when design is based on calculations, alone.

F_{ya} is the actual yield stress of the material, as identified from mill test reports. Where mill test data for the project is not traceable to specific framing elements, the average of mill test data for the project for the given shape may be used. When mill test data for the project is not available, the value of F_{ym}, from table 6-3 may be used.

Z_b is the plastic modulus of the section

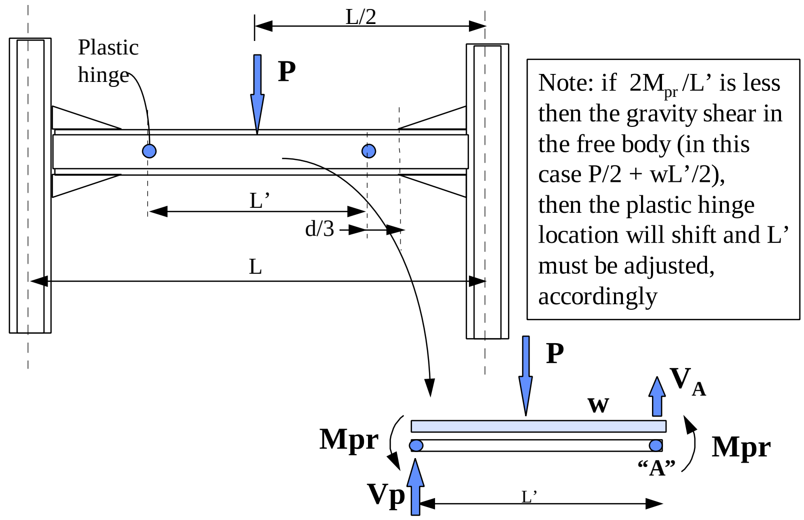
Commentary: The 0.95 factor, in equation 6-1, is used to adjust the yield stress in the beam web, where coupons for mill certification tests are normally extracted, to the value in the beam flange. Beam flanges, being comprised of thicker material, typically have somewhat lower yield strengths than do beam web material.

The factor of 1.1 recommended to account for strain hardening, or other sources of strength above yield, agrees fairly well with available test results. It should be noted that the 1.1 factor could underestimate the over-strength where significant flange buckling does not act as the gradual limit on the connection. Nevertheless, the 1.1 factor seems a reasonable expectation of over-strength considering the complexities involved.

Connection designs that result in excessive strength in the girder connection relative to the column or excessive demands on the column panel zone are not expected to produce superior performance. There is a careful balance that must be maintained between developing connections that provide for an appropriate allowance for girder overstrength and those that arbitrarily increase connection demand in the quest for a "conservative" connection design. The factors suggested above were chosen in an attempt to achieve this balance, and arbitrary increases in these values are not recommended.

6.6.6.3.4 Determine Beam Shear

The shear in the beam, at the location of the plastic hinge should be determined. A free body diagram of that portion of the beam located between plastic hinges is a useful tool for obtaining the shear at each plastic hinge. Figure 6-14 provides an example of such a calculation.

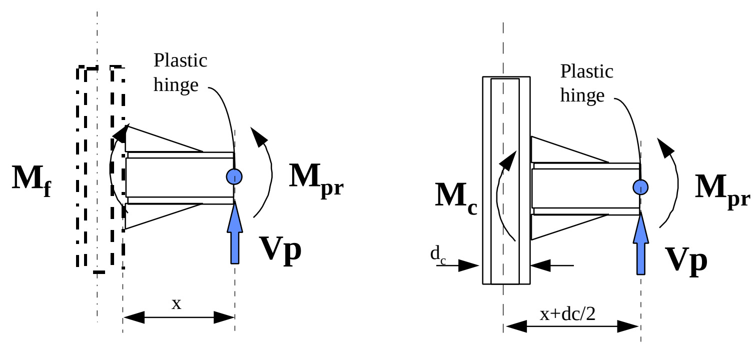


taking the sum of moments about "A" = 0
$Vp = \{Mpr + Mpr + P\ L'/2 + wL'^2/2\}/L'$

Figure 6-14 - Sample Calculation of Shear at Plastic Hinge

6.6.6.3.5 Determine Strength Demands on Connection

In order to complete the design of the connection, including sizing the various plates and joining welds which make up the connection, it is necessary to determine the shear and flexural strength demands at each critical section. These demands may be calculated by taking a free body of that portion of the connection assembly located between the critical section and the plastic hinge. Figure 6-15 demonstrates this procedure for two critical sections, for the beam shown in Figure 6-14.



$M_f = M_{pr} + V_p x$ $M_f = M_{pr} + V_p(x + d_c/2)$

Critical Section at Column Face Critical Section at Column Centerline

Figure 6-15 - Calculation of Demands at Critical Sections

Commentary: Each unique connection configuration may have different critical sections. The vertical plane that passes through the joint between the beam flanges and column (if such joining occurs) will typically define at least one such *critical section, used for designing the joint of the beam flanges to the column, as well as evaluating shear demands on the column panel zone. A second critical section occurs at the center line of the column. Moments calculated at this point are used to check weak beam - strong column conditions. Other critical sections should be selected as appropriate to the connection configuration.*

6.6.6.3.6 Check for Strong Column - Weak Beam Condition

Buildings which form sidesway mechanisms through the formation of plastic hinges in the beams can dissipate more energy than buildings that develop mechanisms consisting primarily of plastic hinges in the columns. Therefore, if an existing building's original design was such that hinging would occur in the beams rather than the columns, care should be taken not to alter this behavior with the addition of connection reinforcement. To determine if the desired strong column - weak beam condition exists, the connection assembly should be checked to determine if the following equation is satisfied:

$$\sum Z_c (F_{yc} - f_a) \Big/ \sum M_c > 1.0 \qquad (6\text{-}2)$$

where:
Z_c is the plastic modulus of the column section above and below the connection
F_{yc} is the minimum specified yield stress for the column above and below
f_a is the axial load in the column above and below
M_c is the moment calculated at the center of the column in accordance with Section 6.6.6.3.5

Commentary: Equation 6-2 is based on the building code provisions for strong column - weak beam design. The building code provisions for evaluating strong column - weak beam conditions presume that the flexural stiffness of the columns above and below the beam are approximately equal. If non-symmetrical connection configurations are used, such as a haunch on the bottom side of the beam, this can result in an uneven distribution of stiffness between the two column segments. In such cases, a plastic analysis should be considered to determine if an undesirable story mechanism is likely to form in the building.

6.6.6.3.7 Check Column Panel Zone

The adequacy of the shear strength of the column panel zone should be checked. For this purpose, the term $0.8\Sigma M_f$ should be substituted for the term $0.8\Sigma M_s$ in *UBC-94* Section 2211.7.2.1 {$0.9\Sigma\phi_b M_p$ in *NEHRP-91* Section 10.10.3.1} repeated below for convenience of reference. M_f is the calculated moment at the face of the column, when the beam mechanism forms, calculated as indicated in Section 6.6.6.3.5, above.

2211.7.2.1 Strength. The panel zone of the joint shall be capable of resisting the shear induced by beam bending moments due to gravity loads plus 1.85 times the prescribed seismic forces, but the shear strength need not exceed that required to develop $0.8\Sigma M_s$ of the girders framing into the column flanges at the joint. The joint panel zone shear strength may be obtained from the following formula:

$$V = 0.55F_y d_c t\left[1 + \frac{3b_c t_{cf}^{\ 2}}{d_b d_c t}\right] \qquad (11\text{-}1)$$

where: b_c = width of column flange
d_b = the depth of the beam (including haunches or cover plates)
d_c = the depth of the column
t = the total thickness of the panel zone including doubler plates
t_{cf} = the thickness of the column flange

6.6.7 Modification Details

There are many potential details that can be used to modify the performance of girder-column joints in existing WSMF structures. Several of these have been tested as part of the SAC Phase 1 effort. While these repair and modification configurations do not represent all potential geometries and the number of replicates is very limited, these tests do provide important insight into the behavior of the modified connection configurations. Figures shown below present conceptual connection configurations that have been subjected to limited testing and have shown an acceptable level of performance.

Reference to laboratory testing is provided for those connection configurations for which research has been reported. However, it should be noted that none of these connections has been tested sufficiently at this time to permit unqualified use of the connection.

The figures provided in the following sections are schematic, indicating the general type of connection configuration being described. When designing connections patterned after the reported test data, the test specimen details included in the references should be reviewed to determine specific details not shown.

The SAC Joint Venture does not endorse or specifically recommend any of the connection details shown in this Section. These are presented only to acquaint the reader with available information on representative testing of different connection configurations that have been performed by various parties.

6.6.7.1 Haunch at Bottom Flange

Figure 6-16 illustrates two alternative configurations of this detail that have been tested (Uang - 1995). The basic concept is to reinforce the connection with the provision of a triangular haunch at the bottom flange. The intended behavior of both configurations is to shift the plastic hinge from the face of the column and to reduce the demand on the CJP weld by increasing the effective depth of the section. In one test, shown on the left of Figure 6-16, the joint between the girder bottom flange and column was cut free, to simulate a condition which might occur if the bottom joint had been damaged, but not repaired. In a second tested configuration, the bottom flange joint was repaired and the top flange was replaced with a locally thickened plate, similar to the detail shown in Figure 6-9.

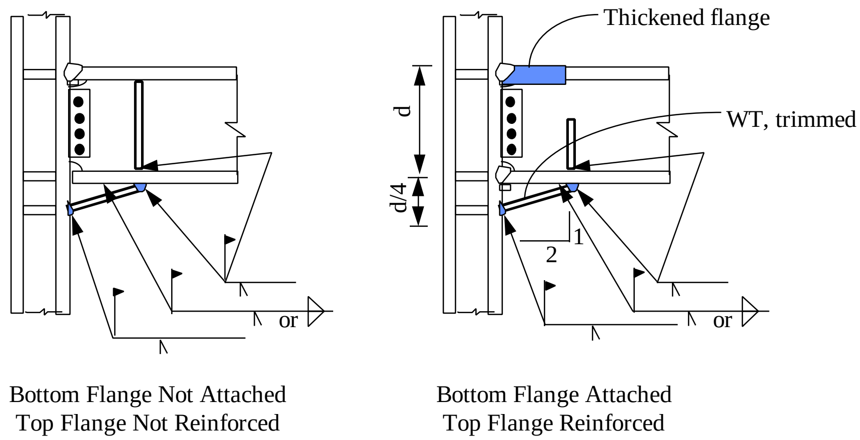


Figure 6-16 - Bottom Haunch Connection Modification

<u>Design Issues</u>: This approach developed acceptable levels of plastic rotation. Acceptable levels of connection strength were also maintained during large inelastic deformations of the plastic hinge. This approach does not require that the top flange be modified, or slab disturbed, unless other conditions require repair of the top flange, as in the detail on the left of Figure 6-16. The bottom flange is generally far more accessible than the top flange because a slab does not have to be removed. In addition, the haunch can be installed at perimeter frames without removal of the exterior building cladding. There did not appear to be any appreciable degradation in performance when the bottom beam flange was not re-welded to the face of the column. Eliminating this additional welding should help reduce the cost of the repair.

Performance is dependent on properly executed complete joint penetration welds at the column face and at the attachment of the haunch to the girder bottom flange. The joint can be subject to through-thickness flaws in the column flange; however, this connection may not be as sensitive to this potential problem because of the significant increase in the effective depth of the beam section which can be achieved. Welding of the bottom haunch requires overhead welding. The skewed groove welds of the haunch flanges to the girder and column flanges may be difficult to execute.

<u>Experimental Results:</u> This approach developed excellent levels of plastic rotation. In Specimen 1, the bottom flange CJP weld was damaged in a prior test but was not repaired: only the bottom haunch was added. During the test of specimen 1, a slowly growing crack developed at the underside of the top flange-web intersection, perhaps exacerbated by significant local buckling of the top flange. Some of the buckling may be attributed to lateral torsional buckling that occurred because the bottom flange was not restrained by a CJP weld. A significant portion of the flexural strength was lost during the cycles of large plastic rotation. In the second specimen, the bottom girder flange weld was intact during the haunch testing, and its performance was significantly improved compared with the first specimen. The test was stopped when significant local buckling led to a slowly growing crack at the beam flange and web intersection. At this time, it appears that repairing damaged bottom flange welds in this configuration can produce

better performance. Acceptable levels of flexural strength were maintained during large inelastic deformations of the plastic hinge for both specimens.

Quantitative Results: No. of specimens tested: 2
Girder Size: W30 x 99
Column Size: W14 x 176
Plastic Rotation achieved-
Specimen 1:0.04 radian (w/o bottom flange weld)
Specimen 2:0.05 radian (with bottom flange weld)

6.6.7.2 Top and Bottom Haunch

Figure 6-17 illustrates the basic geometry of the top and bottom haunch detail. The intended behavior of this modification is to shift the plastic hinge from the face of the column and to reduce the demand on the CJP weld to the column flange by increasing the effective depth of the section. As opposed to the bottom-only haunch, of Section 6.6.7.1, this detail further reduces the demand on all CJP welds and allows for the structural engineer to introduce filler metal with better toughness properties into all critical joints, without necessarily having to remove the top flange CJP weld.

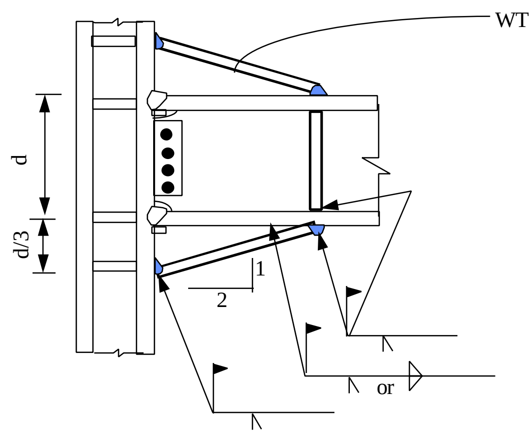


Figure 6-17 - Top and Bottom Haunch Modification Detail

Two specimens for this detail have been tested to date, with excellent results. Possible variations that have not yet been tested include using a shallower haunch at the top flange, substitution of a flat cover plate for the top haunch, and not rewelding either of the original girder flanges to the column, if these have been damaged.

Design Issues: The haunches can be installed at perimeter frames without removal of the exterior building cladding. Performance is dependent on the proper execution of the CJP welds from the haunch to the girder and column flanges, which can be difficult. The joint at the column flange is subject to through-thickness flaws in the column flange, however, due to the additional depth of the section at this joint, and the resulting reduced stresses, this design may not be particularly sensitive to this.

<u>Experimental Results</u>: This approach developed excellent levels of plastic rotation in two specimens. The tests were terminated when fractures across the width of the column flanges developed at the locations of severe buckling in these flanges. Acceptable levels of connection strength were maintained throughout the test.

<u>Quantitative Results</u>: No. of specimens tested: 2
Girder Size: W30 x 99
Column Size: W14 x 176
Plastic Rotation achieved-
Specimen 1:0.07 radian
Specimen 2:0.07 radian

6.6.7.3 Cover Plate Sections

Figure 6-18 illustrates the basic configurations of cover plate connections. The assumption behind the cover plate is that it reduces the demand on the weld at the column flange and shifts the plastic hinge away from the column face. Only the connection with cover plates on the top of the top flange has been tested. There are no quantitative results for cover plates on the bottom side of the top flange, such as might be used in repair. It is likely that thicker plates would be required where the plates are installed on the underside of the top flange. The implications of this deviation from the tested configuration should be considered.

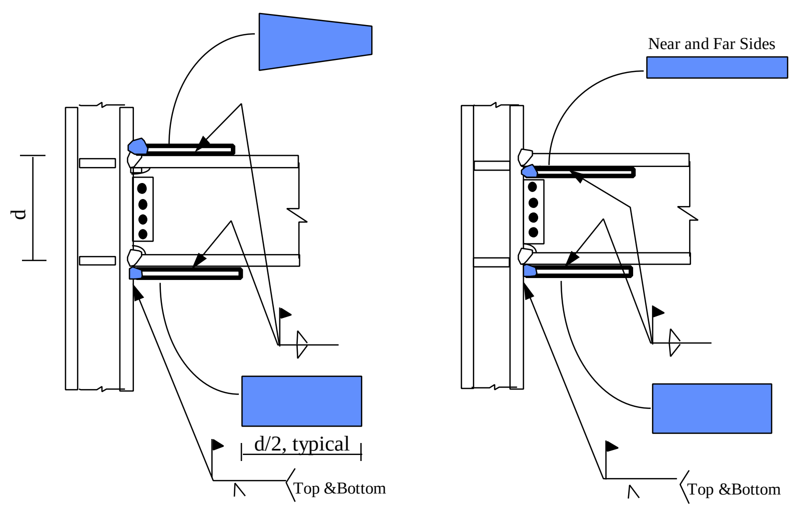


Figure 6-18 - Cover Plate Connection Modification

<u>Design Issues</u>: Approximately eight connections similar that shown in Figure 6-18 have been tested (Engelhardt & Sabol - 1994), and have demonstrated the ability to achieve acceptable levels of plastic rotation provided that the beam flange to column flange welding is correctly executed and through-thickness problems in the column flange are avoided. The option with the top flange cover plate located on top of the flange can be used on perimeter frames where access to the outer side of the beam is restricted by existing building cladding. The option with the cover plate for the top flange located beneath the flange can be installed without requiring modification of the slab. In the figures shown, the bottom cover plate is rectangular, and sized

slightly wider than the beam flange to allow downhand fillet welding of the joint between the two plates. Some configurations using triangular plates at the bottom flange, similar to the top flange have also been tested.

Designers using this detail are cautioned to be mindful of not making cover plates so thick that excessively large welds of the beam flange combination to column flange result. As the cover plates increase in size, the weld size must also increase. Larger welds invariably result in greater shrinkage stresses and increased potential for cracking prior to actual loading. In addition, larger welds will lead to larger heat affected zones in the column flange, a potentially brittle area.

Performance is dependent on properly executed girder flange welds. The joint can be subject to through-thickness failures in the column flange. Access to the top of the top flange requires demolition of the existing slab. Access to the bottom of the top flange requires overhead welding and may be problematic for perimeter frames. Costs are greater than those associated with approaches that concentrate modifications on the bottom flange

Experimental Results: Six of eight connections tested by the University of Texas at Austin were able to achieve plastic rotations of at least 0.025 radians, or better. These tests were performed using heavy column sections which forced nearly all of the plastic deformation into the beam plastic hinge; very little column panel zone deformation occurred. Strength loss at the extreme levels of plastic rotation did not reduce the flexural capacity to less than the plastic moment capacity of the section based on minimum specified yield strength. One specimen achieved plastic rotations of 0.015 radians when a brittle fracture of the CJP weld (type W2 failure) occurred. This may partially be the result of a weld that was not executed in conformance with the specified welding procedure specification. The second unsuccessful test specimen achieved plastic rotations of 0.005 radian when a section of the column flange pulled out (type C2 failure). The successful tests were terminated either when twisting of the specimen threatened to damage the test setup or the maximum stroke of the loading ram was achieved.

Quantitative Results: No. of specimens tested: 8
Girder Size: W36 x 150
Column Size: W14 x 455, and 426
Plastic Rotation achieved-
6 Specimens : >.025 radian to 0.05 radian
1 Specimen: 0.015 radian (W2 failure)
1 Specimen: 0.005 radian (C2 failure)

6.6.7.4 Upstanding Ribs

Figures 6-19 illustrates the basic configuration of connections with upstanding ribs. The assumption behind the rib plate is that it reduces the demand on the weld at the column flange and shifts the plastic hinge from the column face. The figure indicates alternative configurations using either one centered rib, or two spaced ribs on each flange. Test data is available only for the case with two ribs.

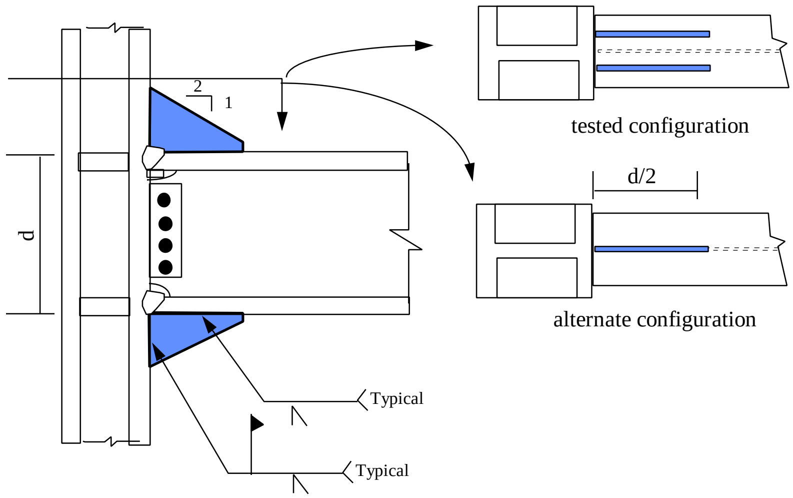


Figure 6-19 - Upstanding Rib Connection Modification

<u>Design Issues</u>: Two connections similar to Figure 6-19, with two spaced ribs at each flange have been tested (Engelhardt & Sabol - 1994), and demonstrated the ability to achieve acceptable levels of plastic rotation provided that the girder flange welding is correctly executed. This modification can be used on perimeter frames where access to the outer side of the girder is restricted by existing building cladding.

Performance is dependent on properly executed girder flange welds. The joint can be subject to through-thickness failures in the column flange. Access to the top of the top flange requires demolition of the existing slab. Access to the bottom of the top flange requires overhead welding and may be problematic for perimeter frames. The size of the specimens tested required the use of two upstanding ribs per flange. This increased the costs significantly above those designs that use only one rib per flange, located above the girder center line. However, limited testing of the design with one rib at the girder centerline, performed as part of a program related to eccentric braced frames, indicated the potential for premature failure of the weld of the rib to the girder at the outstanding edge.

<u>Experimental Results:</u> Two connections have been tested (Engelhardt & Sabol - 1994) using two plates on the top and bottom flanges. The columns used in the test were very heavy and the flanges were able to resist the applied loads from the ribs without distorting. Similar performance might not occur with lighter column sections. In addition, the size of the columns forced all of the plastic deformation into the beam plastic hinge; very little column panel zone deformation occurred. Strength loss at the extreme levels of plastic rotation did not reduce the flexural capacity to less than the plastic moment capacity of the section based on minimum specified yield strength, however, strength loss occurred more quickly than with the cover plated specimens. The tests were terminated when a slow tear of the beam bottom flange occurred at the tips of the ribs.

<u>Quantitative Results</u>: No. of specimens tested: 2
Girder Size: W36 x 150
Column Size: W14 x 426

Plastic Rotation achieved-
2 Specimens : >.025 radian

6.6.7.5 Side-Plate Connections

This approach eliminates loading the column in the through-thickness direction by removing the CJP welds at the girder flange and by shifting the plastic hinge from the column face. The tension and compression forces are transferred from the girder flanges into the column through fillet welds. A mechanism to provide a direct connection between the column panel zone and the beam flanges is required; the difficulty appears to be equalizing the width of the beam and column flanges.

Experimental Results: At least two configurations of side-plated connections have been tested. One set, shown in Figure 6-20, utilized flat bars at the top and bottom girder flanges, to transfer flange forces to the column (Engelhardt & Sabol - 1994). The girder was widened to the width of the column with the use of filler plates. The specimens achieved plastic rotations of 0.015 radians, however, fractures developed within the welds connecting the beam flange to the transfer plates. Failure of the shear tab, and finally the side plates themselves followed the initiation of these fractures. It is believed that the unsuccessful behavior of this particular specimen was related to the method used to increase the width of the beam flange to equal that of the column flange, using a combination of a filler bar and welding. Other approaches that rely on a flat filler plate to transfer the forces may perform better.

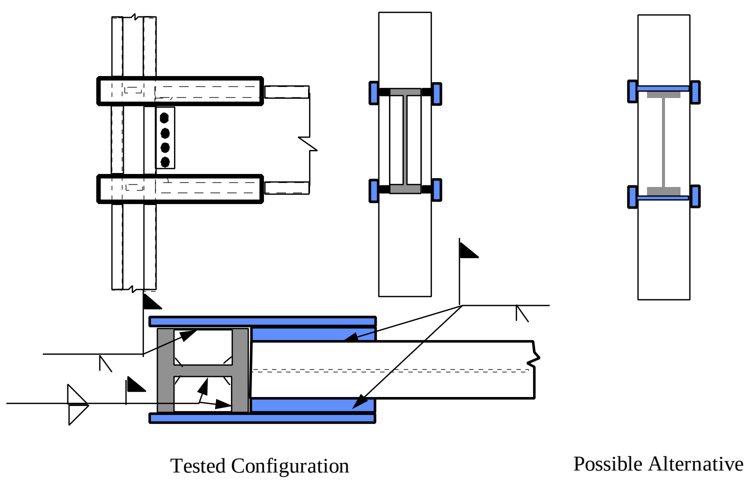
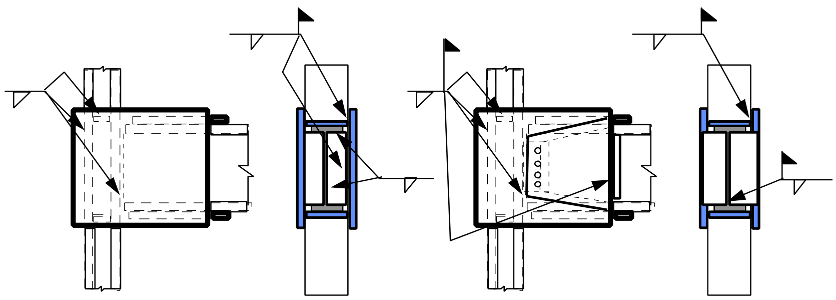


Figure 6-20 - Side Plate Connection Modification

Quantitative Results:

Separate Top & Bottom Side Plates
No. of specimens tested: 2
Girder Size: W36 x 150
Column Size: W14 x 426
Plastic Rotation achieved-
2 Specimens : >.015 radian

A second, proprietary configuration, is shown in Figure 6-21. Three specimens representative of the "new structure" configuration have undergone full-scale testing to date and achieved large plastic rotations. Loss of strength at large plastic rotation demands was comparable to that of other successful connections. No tests have yet been conducted of the repair configuration. The developer of this connection has applied for US and foreign patents. Further information on technical data for this configuration, and license fees, may be obtained from the developer.



New Building Configuration Repair Configuration

WARNING: The information presented in this figure is PROPRIETARY. US and Foreign Patents have been applied for. Use of this information is strictly prohibited except as authorized in writing by the developer. Violators shall be prosecuted in accordance with US and Foreign Patent Intellectual Property Laws.

Figure 6-21 - Proprietary Side Plate Connection Modification

Design Issues: Testing of three prototype specimens (Uang & Latham - 1995) indicates that this connection has the ability to achieve very satisfactory levels of plastic rotation without relying on sensitive CJP welds between the column and girder flanges, or requiring specification of notch-tough weld material. The elimination of the through-thickness loading of the flange may result in higher levels of connection reliability. Due to the exclusive use of fillet welds, special inspection requirements for welding and bolting can be reduced significantly with this connection.

This connection is proprietary (patent pending) and not in the public domain. It has not been tested in a repair condition. Access to the top of the top flange of the girder might require demolition of the existing slab. The cost of the connection may be greater than some of the other modification methods discussed above; however, this cost differential may not be as great on double-sided connections because much of the cost is associated with the side plates which are similar for both single-sided and double-sided connections. Publicly bid projects may have to develop performance specifications to permit other connections to be considered for use unless a strong case for sole-sourcing the connection can be made.

Quantitative Results:

No. of specimens tested: 3
Girder Size: W36 x 150
Column Size: W14 x 426

Plastic Rotation achieved-

3 Specimens : >.042 to 0.06 radian

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