6. POSTEARTHQUAKE REPAIR

6.1 Scope

This section provides criteria for structural repair of earthquake damage. *Repair* constitutes any measures taken to restore earthquake damaged elements of the building, including individual members or their connections, or the building as a whole, to their original configuration, strength, stiffness and deformation capacity. It does not include routine correction of non-conforming conditions resulting from the original construction or upgrades intended to result in improvement in future seismic performance of the building. Repair must typically be performed under a building permit, requiring submittal, to the building department of construction documents, inspection and testing reports.

Sections 6.2 through 6.3 provide recommended methods of repair for various types of damage. These recommendations are not intended to be used for the routine repair of construction non-conformance commonly encountered in fabrication and erection work. Industry standard practices are acceptable for such repairs. Recommendations for assessment of the seismic performance capability of existing buildings and upgrade of buildings to improve performance capability may be found in a companion publication, *FEMA-351, Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings*.

Commentary: Based on the observed behavior of actual buildings in the Northridge earthquake, as well as recent test data, welded steel moment frame buildings constructed with the typical detailing and construction practice prevalent prior to 1994 do not have the deformation capacity they were presumed to possess at the time of their design and therefore present significantly higher risks than was originally thought. When these buildings are damaged or have latent construction defects, this risk is higher still.

Based on limited testing, it is believed that the repair recommendations contained in these Recommended Criteria can be effective in restoring a building's pre-earthquake condition, and to the extent that the detailing, workmanship and materials of repair work are superior to the original construction, provide some marginal improvement in seismic performance capability. 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 upgraded, 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 upgraded to provide improved future performance. Seismic upgrade can consist of local reinforcement of individual moment connections, column splices and other critical connections, as well as alteration of the basic lateral-force-resisting characteristics of the structure through addition, for example, of braced frames, shear walls, base isolation, and energy dissipation devices. Performance *evaluation and structural upgrade are beyond the scope of these Recommended Criteria. Criteria for performance evaluation and structural upgrade may be found in a companion document, FEMA-351, Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings.*

6.2 Shoring and Temporary Bracing

6.2.1 Investigation

Prior to engaging in repair activity, the structural engineer should investigate the entire building and perform an evaluation to determine if any imminent collapse or life-safety hazard conditions exist and to determine if the structure as a whole provides adequate stability to safeguard life during the repair process. The Level 2 evaluation process of Chapter 5 is one method of confirming both the building's global structural stability, and the ability of individual connections to withstand ground shaking. Where hazardous conditions or lack of stability are detected, shoring and or temporary bracing should be provided 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 that 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 damage 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 developed. 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 contract documents may require that the contractor submit a shoring design, prepared by another registered structural engineer, 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 recommended criteria, for reference, only.

Commentary: Examples of repair details are provided for some classes of damage, based on approaches successfully performed in the field following the 1994 Northridge earthquake. 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.

In typical practice for construction of new buildings, the selection of means and methods used to construct design details are typically left to the contractor. In structural repair work, the members are typically under greater load and also restraint during the fabrication and erection process than is common in new construction. Therefore, the typical construction practices may not be appropriate and many contractors may not have the knowledge or experience to select appropriate methods for repair work. As a result, much greater specification of means and methods is recommended than is common in new construction. Although it is recommended that the engineer provide such specification as part of the construction documents, the engineer should also be open to suggestions for alternative procedures if the contractor desires to submit such procedures. If there is doubt as to the ability of alternative procedures to provide acceptable construction, a full-scale mock-up test of the proposed procedure should be considered.

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:

- replacement of damaged portions of base metal (i.e. column and beam section),
- replacement of damaged connection elements,
- replacement of connection welds, or
- repairs to portions of any of the aforementioned components.

Any or all of these techniques may be appropriate. The approaches used should consider adjacent structural components that may be affected by the repair or the effects of the repair.

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

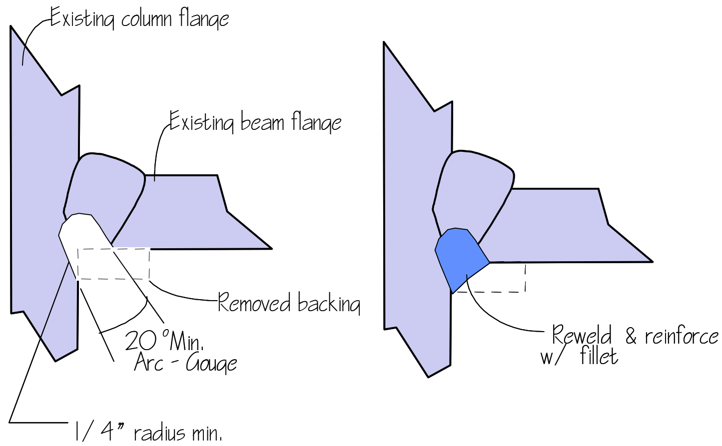
6.3.2 Weld Fractures - Type W Damage

Prior to repair of fractures or rejectable defects in welds, sufficient material should be removed to completely eliminate any existing discontinuity or defect in the weld metal and if applicable, adjacent base material. Nondestructive Testing should be used to determine the extent of fracture or defect present 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 back gouging to eliminate unsound weld material and replacement of 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 an appropriate Welding Procedure Specification (WPS) and the project specifications. Weld backing, existing end dams, and weld tabs should be removed from all welds that are being repaired. End dams should not be permitted in new work. After backing and tab elements are removed, the weld root should be back gouged to sound material, re-welded 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 the 1989 *AISC ASD Steel Construction Manual*, ninth edition, page 5-69). As an example, E7018 shielded metal arc welding (SMAW) electrodes should not be used to weld over self-shielded flux cored arc welding (FCAW-S) deposits, unless appropriate precautions are taken (*FEMA 355b*). Typically, three to four passes of E7018 or similar notch tough filler metal should be deposited to ensure that the underlying FCAW-S filler metal has not degraded the overlying notch tough filler metal. 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

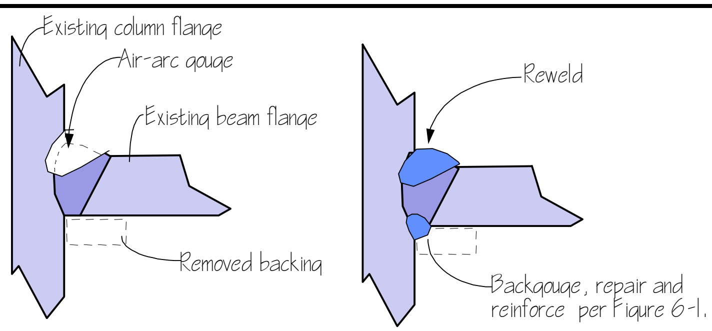
Damage or Defect Class	Figure
Rejectable defects at weld root	Figure 6-1, Figure 6-2
W2	Figure 6-3
W3	Figure 6-3
W4	Figure 6-3
W5	Figure 6-3



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 ¼".
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 that 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.

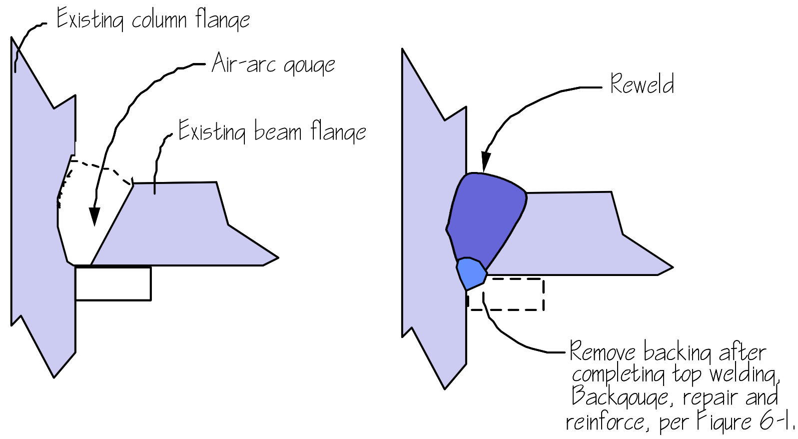
Figure 6-1 Gouge and Re-weld of Root Defect or Damage



Notes:

1. Remove the entire fracture plus 1/8" of sound metal beyond each end.
2. For additional notes, refer to Figure 6-1

Figure 6-2 Gouge and Re-weld of Fractured Weld



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

Figure 6-3 Backgouge and Reweld Repair

Commentary: Flux-cored arc-welding (FCAW-S) 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-S 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-S and subsequent repair welds are made with shielded-metal arc-welding, SMAW (stick), using E7018, for example, the SMAW arc will penetrate into the FCAW-S 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-S material.

Various types of FCAW-S electrodes may be mixed one with the other without potentially harmful effect. Further, notch tough FCAW-S may be used to weld over other types of weld deposits without potentially harmful interaction. The structural engineer could specify all repairs on FCAW-S deposits be made with FCAW-S. Alternatively, intermixing of FCAW-S and other processes could be permitted provided the subsequent composition is demonstrated to meet material specification requirement, or adequate layers of reinforcing notch-tough filler metal are installed to avoid segregation (FEMA 355B).

The recommendations contained in Chapters 3, 4, and 5 for inspection and evaluation of damaged buildings do not require extensive nondestructive examination of welds to detect defects or fractures that are not detectable by visual inspection but are rejectable under the AWS D1.1 provisions.

Nevertheless, it is likely that in the course of performing inspection and repair work, some such rejectable conditions will be found. It is recommended that any such detected conditions be repaired as part of the overall building repair program, as their presence in welds make the welds significantly more vulnerable to future fracturing under loading, particularly if the welds are composed of material with limited notch toughness.

In the past, there has been considerable disagreement as to whether or not small cracks and defects at the root of a weld are earthquake damage or not. Proper observation by knowledgeable persons can reveal whether a root defect is a slag inclusion or lack of fusion, both conditions relating to the original construction, or an actual crack. It should be noted that cracks may not necessarily be caused by the building's earthquake response. Some cracking invariably occurs in structures during the erection process as a result of residual stress conditions and thermal stresses. It is almost impossible to distinguish such cracks from those caused by an earthquake. Through detailed examination of the fracture surface for evidence of oxidation or other signs of age it may be possible to obtain clues as to when a crack initiated. Many researchers believe that the low toughness weld metal commonly used in construction prior to 1994 was incapable of arresting an earthquake induced fracture, once it initiated in a joint and that small cracks that do not penetrate through the metal are unlikely to be earthquake related. However, there have been reports from laboratory testing that indicate that small cracks do form in the weld metal and arrest prior to development of unstable fracture conditions, even in low-toughness weld metals. Therefore, without detailed examination of an individual fracture by knowledgeable individuals, no conclusive statement can be made as to whether weld cracking is earthquake induced.

6.3.3 Column Fractures - Types C1 to C5 and P1 to 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, magnetic particle testing (MT) and/or Liquid Dye Penetrant testing (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. Table 6-2 indicates 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 and Figures 6-9 and 6-10 for repair of girders, or if access is restricted, as an alternative beam repair method.

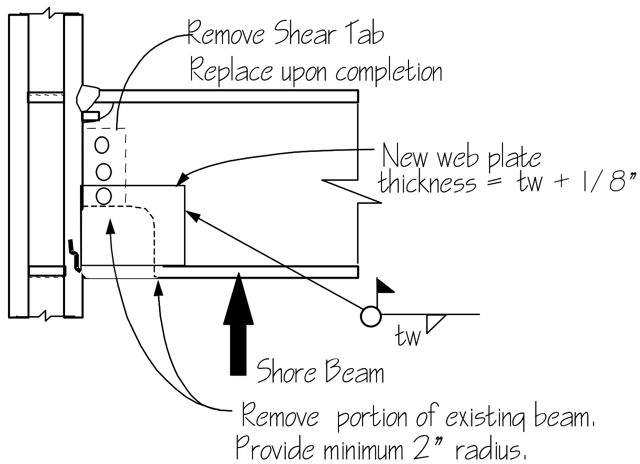


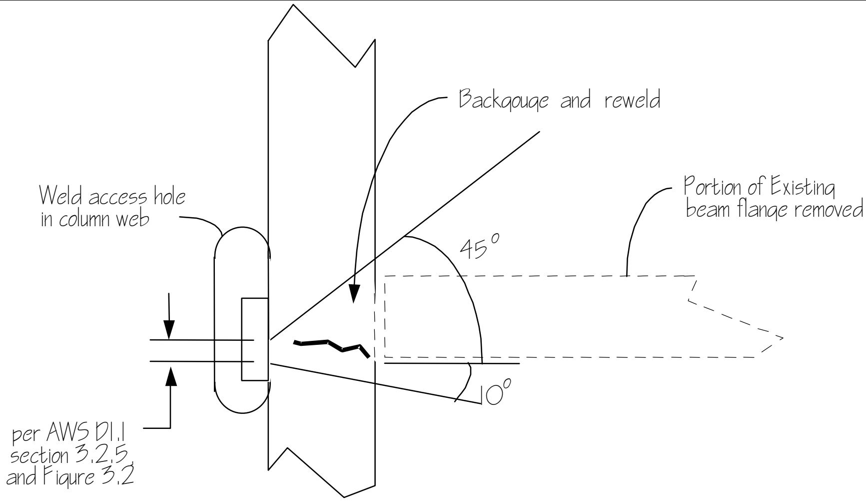
Figure 6-4 Temporary Removal of Beam Section for Access

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	Figures 2-3, 6-4, 6-5
C2	Figures 2-3, 6-4, 6-6
C3	Figures 2-3, 6-4, 6-5
C4	Figures 2-3, 6-4, 6-5
C5	Figures 2-3, 6-4, 6-6
P1	Figure 2-6; remove, prepare, replace
P2	Figure 2-6; arc-gouge and reweld
P4	Figure 2-6; arc-gouge and reweld
P5	Figures 2-6, 6-7
P6	Figures 2-6, 6-7
P7	Figures 2-6, 6-7
P8	Figures 2-6, 6-8

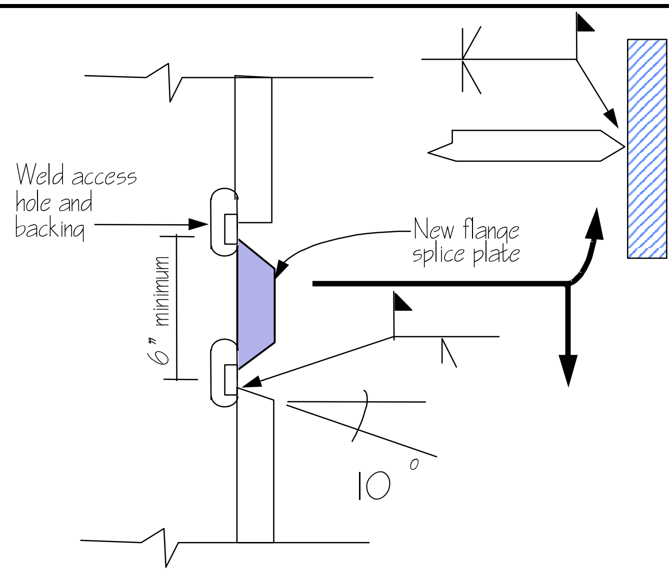
Commentary: Special attention should be given to conditions where more than 20% of the column cross section will be removed at one time, as special temporary shoring may be warranted. In addition, care should be taken when applying heat to a flange or web containing a fracture, as fractures have been 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.



Notes:

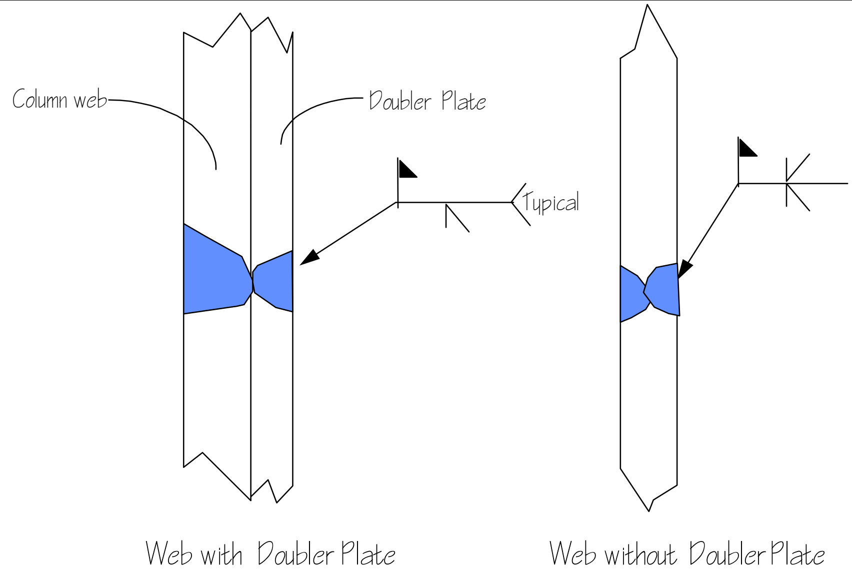
1. Investigate extent of fracture by UT to confirm that fractures are contained with the 45 degree angle zone of a standard pre-qualified CJP groove weld as defined in Figure 2.4, AWS D1.1, Joint Designation B-U4a-G
2. Provide 10^o bevel on lower flange plate, to channel slag out of joint.
3. Grind all 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"/+¼". 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^o, to channel slag out of joint.

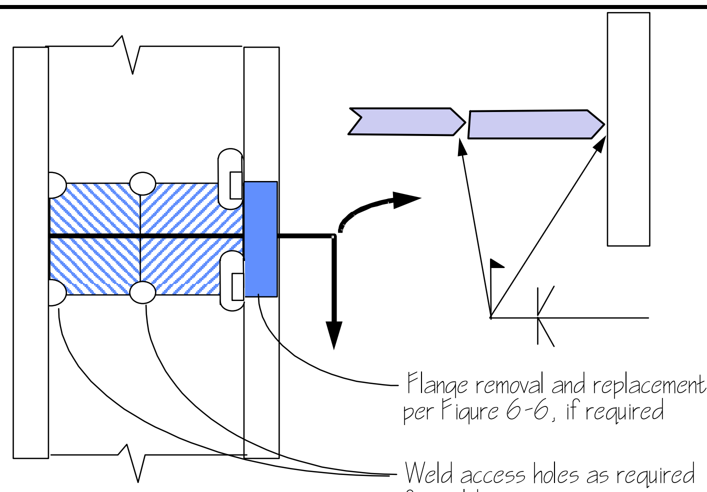
Figure 6-6 Replacement of Column Flange Repair



Notes:

1. Prepare fractured section of doubler by air-arc gouging, grinding and rewelding, using web as backing.
2. Prepare fractured section of web by air-arc gouging, grinding and rewelding, 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 –0"/+ ").

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

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 (see Figures 6-5 and 6-6). Also, the addition of flange plates to the outside face of each flange may be considered.

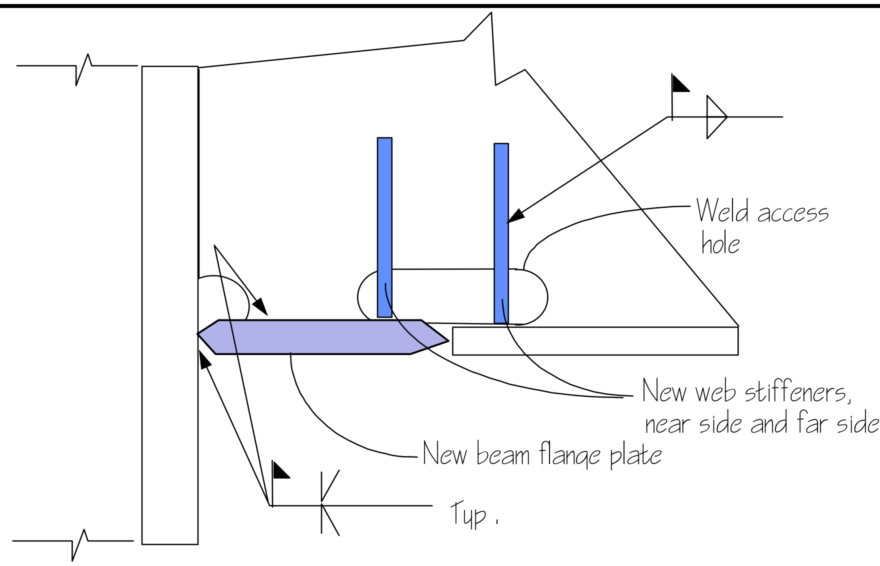
6.3.5 Girder Flange Fractures - Type G3 to G5

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.

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.

In the case of restricted access on one side of the beam (facade interference) it may be advantageous to make the plate narrower than the beam flange and perform all welding overhead. A CJP weld and fillet weld should be used to connect the plate to the column flange and beam flange, respectively.

It is recommended that a double bevel joint be utilized in replacing the removed plate to eliminate the need for backing, consequently also eliminating the need for removal of the backing upon joint completion. A suggested joint detail is a B-U3/TC-U5, per AWS D1.1, with 1/3 t_{flange} to 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 beam flange butt welds, but should be removed following joint completion. 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.*



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

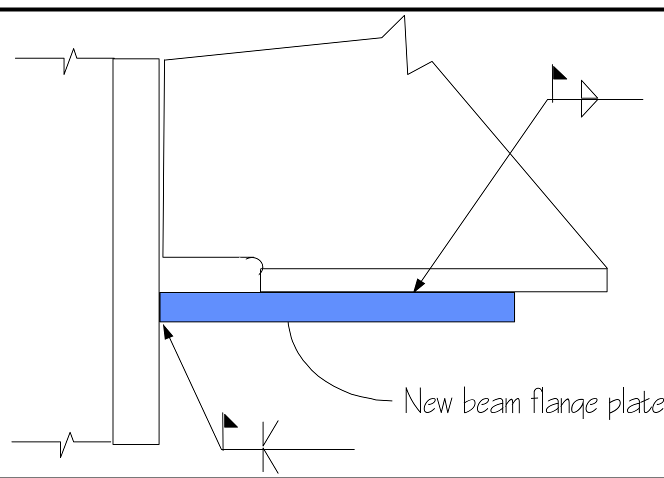
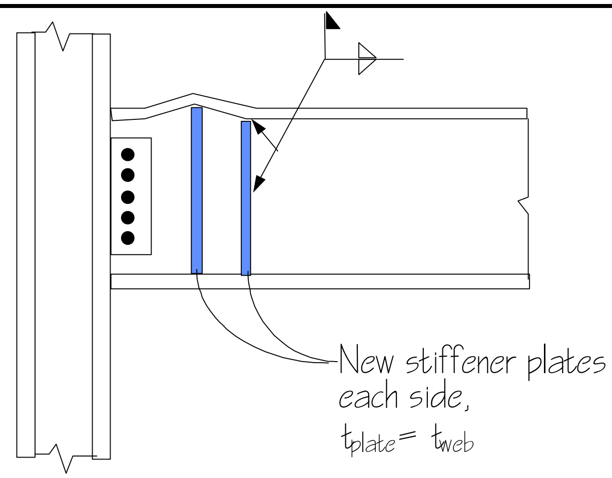


Figure 6-10 Alternative Beam Flange Plate Replacement

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 1989 or 1994 *AISC Manual of Steel Construction* 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 the 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; Shonafelt and Horn, 1984)

6.3.8 Gravity Connections

Connections not part of the lateral force-resisting system may also be found to require repair due to excessive rotation or demand caused by distress of the lateral-force-resisting system. 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-force-resisting system should be repaired in a similar manner, with special consideration given to the nature and significance of the overall structural damage.

Commentary: Testing of typical gravity shear connections conducted as part of the research performed in support of the development of these Recommended Criteria indicates that shear tab connections are capable of sustaining very large rotation demands without compromise of their gravity load carrying capacity (FEMA-355D). These connections tend to degrade in strength only when the imposed rotation becomes large enough to induce contact between the beam flange and the adjacent member. Once this contact occurs, rotational resistance of the connection stiffens substantially with large forces generated both through bearing of the beam flange against the adjacent member and as an axial force transmitted through the shear tab. The resulting forces can compromise, the bolts, the weld of the shear tab to the supporting member or the shear tab itself. Such behavior has not occurred in laboratory testing until connection rotations in excess of 0.1 radians were achieved.

6.3.9 Reuse of Bolts

Bolts in a connection displaying bolt damage or plate slippage should not be re-used, except as indicated herein. As indicated in the *AISC Specification for Structural Joints using ASTM A325 or A490 Bolts* (AISC, 1985), A490 bolts and galvanized A325 bolts should not be re-tightened and re-used under any circumstances. Other A325 bolts may be reused if determined to be in good condition. Touching up or re-tightening 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*.

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 A490 bolts and galvanized ASTM A325 bolts become slightly damaged when tightened, and consequently, should not be reused. To determine if an ungalvanized ASTM A325 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 Specifications

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. 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 any requirements that are mandated by the project specification. Two categories of qualified welding procedures are given in *AWS D1.1*. These are pre-qualified welding procedures and qualified-by-test welding procedures. Regardless of the type of qualification of a proposed welding procedure, a WPS should be prepared by the contractor and reviewed by the structural engineer responsible for the repairs.

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. 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 during the construction of at least the first weld repair. This observation should be made by a qualified welding inspector or 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 that is more challenging than work in which the welder has previously demonstrated competence. This will allow the welder to become familiar with atypical welds, and will give the inspector the opportunity to observe clearly 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. In a complete mock-up, all welding positions and types of weld would be experienced, thus showing the welder capable of both completing successfully the weld in all required positions, and applying all heating requirements.

Commentary: The structural engineer may, at his or her discretion, require joint mock-ups to be performed for specific types of repair work as part of the project specifications. This practice is recommended where repair work must be made under conditions of unusual or restricted access, under conditions of high restraint, or for any joint that is not routinely performed in the industry.

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, and fractured columns should be indicated on the drawings as appropriate to guard against structural failure and 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 possess significant ability to participate in the structure's lateral-force-resisting system. Consideration should be given to limiting the total number of connections being repaired at any one time, as the overall lateral-force 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 proactive role in determining this.

6.5 Execution

6.5.1 General

FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications, provides recommended general requirements to be included in specifications for repair. The following are of particular importance:

- Strict enforcement of the welding requirements in *AWS D1.1* adopted and modified by the building code.
- Implementation of the special inspection requirements in the *1997 NEHRP Recommended Provisions for New Buildings* and *AWS D1.1,* as well as such other requirements enforced by the local Building Department. 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.
- Require the fabricator to prepare and submit a WPS with at least the information required by *AWS D1.1,* as discussed in Section 6.5.4.

- Welding electrodes for welded joints in severe service and with significant consequences of failures should be capable of depositing weld metal with a minimum notch toughness of 40 ft-lbs at the anticipated service temperature and 20 ft-lbs at 0^{o}F. Refer to *FEMA-353*. Joints in this category include all complete joint penetration groove welds in beam and column flanges and webs, welds of continuity plates to column flanges and similar elements subject to large cyclic stresses at or near plastic levels.
- 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 per *AWS D1.1*, Section 8.15.1. Surface imperfections less than 1/16 inch in dimension should be removed by grinding. Deeper gouges, areas of lack of fusion, and slag inclusions, for example, should be removed by gouging or grinding and rewelding following the procedures outlined above.
- Weld tabs should conform to the requirements of *AWS D1.1*. End dams should not be permitted.
- 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.

6.5.2 Removal of Backing

Prior to removing weld backing on existing joints, 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 the contractor should propose the method(s) that will be used to remove the weld backing, back gouge to sound metal and when during this process preheat will be applied.

Although project conditions may vary, steel backing may be removed either by 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 magnetic particle testing (MT) to determine if any linear indications remain. If the area is free of indications the area may then be re-welded. 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.

> *Commentary: Only removal of backing from the bottom-beam-flange-to-column joint is recommended. Removal of the weld backing from the top flange may be difficult, particularly along perimeter frames where access to the outer side is*

restricted. Tests conducted to date have not been conclusive with regard to the benefit of top flange weld backing removal, and therefore, this is not generally recommended. There is no direct evidence that removal of weld backing from continuity plates in the column panel zone is necessary.

6.5.3 Removal of Weld Tabs

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. 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 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.

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

6.5.4 Defect Removal

Any rejectable 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 ultrasonic testing (UT) results which describe the location depth and extent of the defect(s).

If defect removal extends into the theoretical weld section, 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.

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 that 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 individual 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.

6.5.5 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. The reinforcing fillet should have a size equal to one-quarter of the girder flange thickness. It need not exceed 3/8 inch (see Note J, Figure 3.4 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 inch 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 inch 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 in the vicinity of 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 inch should be removed by grinding. Deeper gouges, areas of lack of fusion, and slag inclusions, for example, should be removed by gouging or grinding and rewelding following the procedures outlined above.

6.5.6 Weld Repair (Types W2, or W3 and Defects)

When 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. Similar procedures should be followed when making repairs to defects at weld roots.

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.

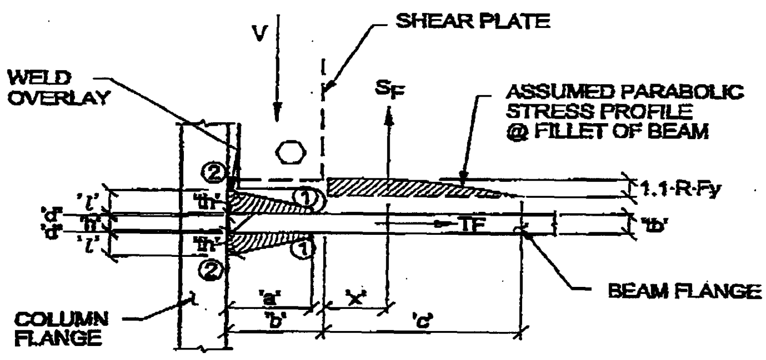
Repair of cracks and defects in welds may warrant total removal of the original weld, particularly if multiple cracks are present. If the entire weld plus some base metal is removed, care must be taken not to exceed the root opening and bevel limits of *AWS D1.1* unless a qualified by test WPS is used. 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.

6.5.7 Weld Overlays

An alternative approach that can be considered when repairing or reinforcing pre-Northridge beam-flange to column-flange welded connections is to apply a weld overlay. This procedure is suitable for repairing beam-flange welds that have been classified as having rejectable weld defects, as opposed to fractures. This application consists of encapsulating the existing low toughness weld metal with material with a higher notch toughness. The high notch toughness weld metal overlay is very resistant to fracture initiation from surface discontinuities within the range of defects that would go undetected, effectively converts surface defects and small fractures in the existing CJP weld into internal defects, and further, provides local reinforcement of the joint for resistance of applied stresses. This repair approach was developed by an independent task group of engineers and researchers in the Los Angeles area, following the 1994 Northridge earthquake. A schematic arrangement of a weld overlay is shown in Figure 6-12.



Nomenclature:

F_{yb} = Minimum specified yield stress of beam (force / unit area)

F_{yc} = Minimum specified yield stress of column (force / unit area)

V = Total Shear acting on connection based upon nominal beam flexural strength at the plastic hinge location (i.e., $\frac{1.1R_y 2M_p}{L_c} + V_{gravity}$, where L_c = clear distance between column flanges)

R_y = Ratio of Expected Yield Strength F_{yc} to the minimum specified yield strength F_{yb}

S_F = Total shear applied to each flange. (force)

T_F = Total tension applied to each flange. (force)

h = Height of flaw assumed over full width of flange. (dimension)

Figure 6-12 Weld Overlay Repair of Beam Flange to Column Flange Joint

The design of the weld overlay is based on the premise that, in addition to transferring the flange flexural force to the column, the beam shear force must also be transferred through the beam flange weld. Based on physical testing of typical connections, supplemented by finite element analysis the stress distribution across the beam-flange is assumed to be parabolic as shown in Figure 6-13. Design procedures have been developed for two overlay conditions: Class A, which requires that the overlay take the full connection demand; and Class C, which assumes that the remaining existing weld has 50% of its original design capacity remaining. The throat thickness of the weld overlay, which may be regarded as an elongated fillet weld, is determined from geometric considerations similar to standard fillet welds.

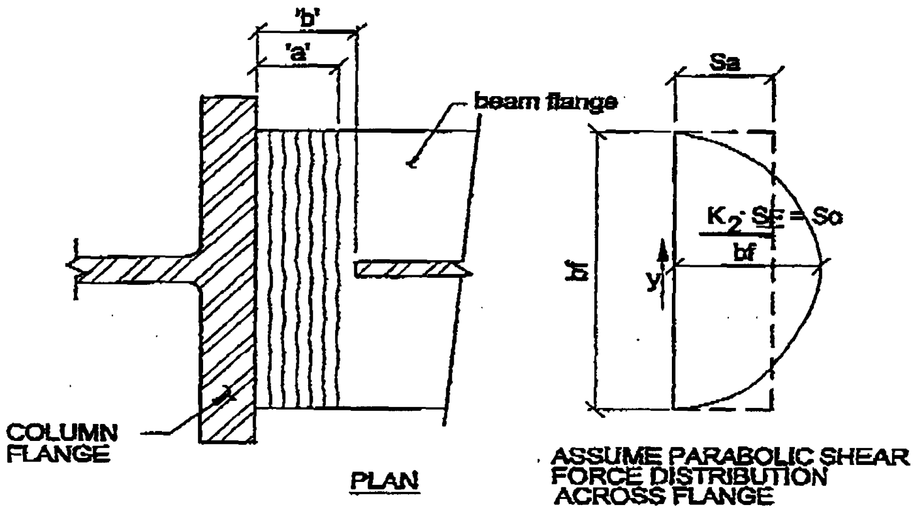


Figure 6-13 Plan View and Assumed Stress Distribution for Weld Overlay Design

Commentary: Independent research into the performance of weld overlay repairs indicates that beam-column connections repaired using this technique are substantially more rugged than typical unreinforced connections employing low toughness weld metals. Refer to Anderson, et al. (2000) for more detailed information on this technique.

6.5.8 Column Flange Repairs - Type C2

Damage type C2 is a pullout type failure of the column flange material. The failure surface 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 (buttering). 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 magnetic particle testing and ultrasonic testing.