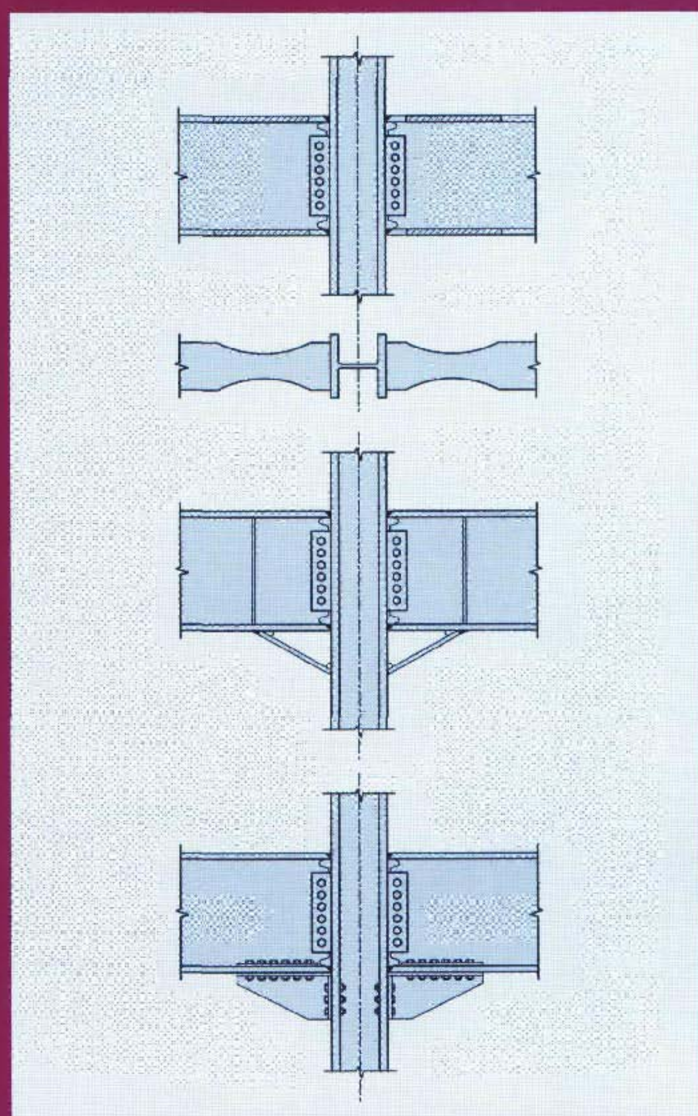
Steel Design Guide Series

12

Modification of Existing Welded Steel Moment Frame Connections for Seismic Resistance



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Modification of Existing
Welded Steel Moment Frame
Connections for Seismic Resistance

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PREFACE

The Congressional emergency appropriation resulting from the January 17, 1994, Northridge earthquake provided the Building and Fire Research Laboratory (BFRL) at the National Institute of Standards and Technology (NIST) an opportunity to expand its activities in earthquake engineering under the National Earthquake Hazard Reduction Program (NEHRP). In addition to the post-earthquake reconnaissance, BFRL focused its efforts primarily on post-earthquake fire and lifelines and on moment-resisting steel frames.

In the area of moment-resisting steel frames damaged in the Northridge earthquake, BFRL, working with practicing engineers, conducted a survey and assessment of damaged steel buildings and jointly funded the SAC (Structural Engineers Association of California, Applied Technology Council, and California Universities for Research in Earthquake Engineering) Invitational Workshop on Steel Seismic Issues in September 1994. Forming a joint university, industry, and government partnership, BFRL initiated an effort to address the problem of the rehabilitation of existing buildings to improve their seismic resistance in future earthquakes. This design guideline is a result of that joint effort.

BFRL is the national laboratory dedicated to enhancing the competitiveness of U.S. industry and public safety by developing performance prediction methods, measurement technologies, and technical advances needed to assure the life cycle quality and economy of constructed facilities. The research conducted as part of this industry, university, and government partnership and the resulting recommendations provided herein are intended to fulfill, in part, this mission.

This design guide has undergone extensive review by the AISC Committee on Manuals and Textbooks; the AISC Committee on Specifications, TC 9—Seismic Design; the AISC Committee on Research; the SAC Project Oversight Committee; and the SAC Project Management Committee. The input and suggestions from all those who contributed are greatly appreciated.

Chapter 1
INTRODUCTION

The January 17, 1994 Northridge Earthquake caused brittle fractures in the beam-to-column connections of certain welded steel moment frame (WSMF) structures (Youssef et al. 1995). No members or buildings collapsed as a result of the connection failures and no lives were lost. Nevertheless, the occurrence of these connection fractures has resulted in changes to the design and construction of steel moment frames. Existing structures incorporating pre-Northridge[1] practices may warrant re-evaluation in light of the fractures referenced above.

The work described herein addresses possible design modifications to the WSMF connections utilized in pre-Northridge structures to enhance seismic performance.

1.1 Background

Seismic design of WSMF construction is based on the assumption that, in a severe earthquake, frame members will be stressed beyond the elastic limit. Inelastic action is permitted in frame members (normally beams or girders) because it is presumed that they will behave in a ductile manner thereby dissipating energy. It is intended that welds and bolts, being considerably less ductile, will not fracture. Thus, the design philosophy requires that sufficient strength be provided in the connection to allow the beam and/or column panel zones to yield and deform inelastically (SEAOC 1990). The beam-to-column moment connections should be designed, therefore, for either the strength of the beam in flexure or the moment corresponding to the joint panel zone shear strength.

The Uniform Building Code, or UBC (ICBO 1994) is adopted by nearly all California jurisdictions as the standard for seismic design. From 1988 to 1994 the UBC prescribed a beam-to-column connection that was deemed to satisfy the above strength requirements. This "prescribed" detail requires the beam flanges to be welded to the column using complete joint penetration (CJP) groove welds. The beam web connection may be made by either welding directly to the column or by bolting to a shear tab which in turn is welded to the column. A version of this prescribed detail is shown in Figure 1.1. Although this connection

[1]The term "pre-Northridge" is used to indicate design, detailing or construction practices in common use prior to the Northridge Earthquake.

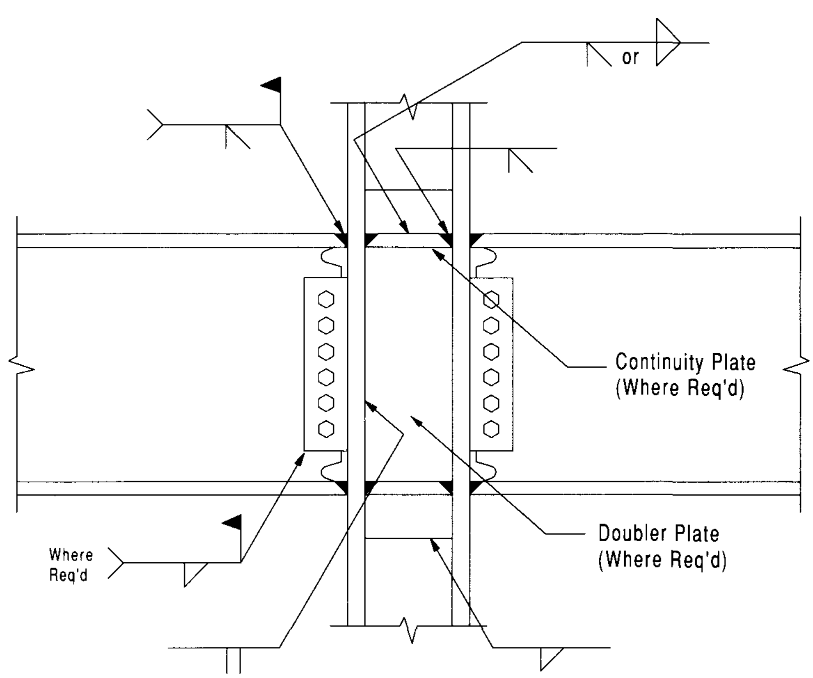


Figure 1.1 Prescribed Welded Beam-to-Column Moment Connection (Pre-Northridge)

detail was first prescribed by the UBC in 1988, it has been widely used since the early 1970's.

The fractures of "prescribed" moment connections in the Northridge Earthquake exhibited a variety of origins and paths. In general, fracture was found to initiate at the root of the beam flange CJP weld and propagate through either the beam flange, the column flange, or the weld itself. In some instances, fracture extended through the column flange and into the column web. The steel backing, which was generally left in place, produced a mechanical notch at the weld root. Fractures often initiated from weld defects (incomplete fusion) in the root pass which were contiguous with the notch introduced by the weld backing. A schematic of a typical fracture path is shown in Figure 1.2. Brittle fracture in steel depends upon the fracture toughness of the material, the applied stress, and size and shape of an initiating defect. A fracture analysis, based upon measured fracture toughness and measured weld defect sizes (Kaufmann et al. 1997), revealed that brittle fracture would occur at a stress level roughly in the range of the nominal yield stress of the beam.

The poor performance of pre-Northridge moment connections was verified in laboratory testing conducted under SAC[2] Program to Reduce Earthquake Hazards in Steel Moment-Resisting Frame Structures (Phase 1) (SAC 1996). Cyclic loading tests were conducted on 12 specimens constructed with W30X99 and W36x150 beams. These specimens used connection details and welding practices in common use prior to the Northridge Earthquake. Most of the 12 specimens failed in a brittle manner with little or no ductility. The average beam plastic rotation developed by these 12 specimens was approximately 0.005 radian. A number of specimens failed at zero plastic rotation, and at a moment well below the plastic moment of the beam. Figure 1.3 shows the results of one of these tests conducted on a W36x 150 beam.

[2]SAC is a Joint Venture formed by the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), and the California Universities for Research in Earthquake Engineering (CUREe).

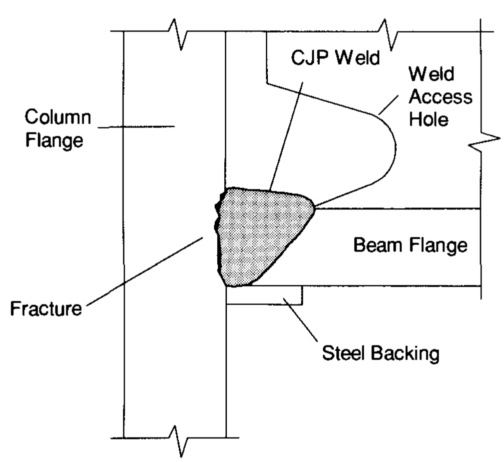


Figure 1.2 Typical Fracture Path

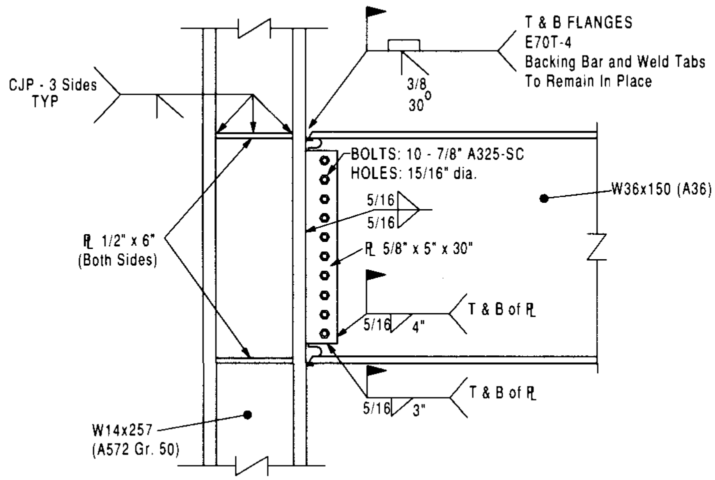
1.2 Factors Contributing to Connection Failures

Brittle fracture will occur when the applied stress intensity, which can be computed from the applied stress and the size and character of the initiating defect, exceeds the critical stress intensity for the material. The critical stress intensity is in turn a function of the fracture toughness of the material. In the fractures that occurred in WSMF construction as a result of the Northridge Earthquake, several contributing factors were observed which relate to the fracture toughness of the materials, size and location of defects, and magnitude of applied stress. These factors are discussed here.

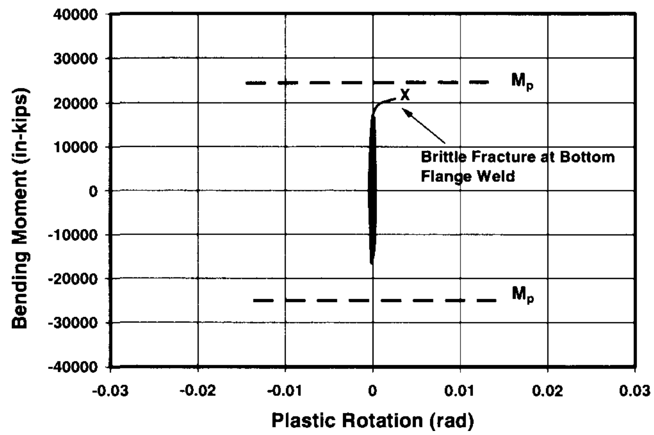
The self-shielded flux cored arc welding (FCAW) process is widely used for the CJP flange welds in WSMF construction. Electrodes in common use prior to the Northridge earthquake are not rated for notch toughness. Testing of welds samples removed from several buildings that experienced fractures in the Northridge earthquake revealed Charpy V-notch (CVN) toughness frequently on the order of 5 ft-lb to 10 ft-lb at 70°F (Kaufmann 1997). Additionally, weld toughness may have been adversely affected by such practices as running the weld "hot" to achieve higher deposition rates, a practice which is not in conformance with the weld wire manufacturer's recommendations.

The practice of leaving the steel backing in place introduces a mechanical notch at the root of the flange weld joint as shown in Figure 1.2. Also, weld defects in the root pass, being difficult to detect using ultrasonic inspection, may not have been characterized as "rejectable" and therefore were not repaired. Further, the use of "end dams" in lieu of weld tabs was widespread.

The weld joining the beam flange to the face of the relatively thick column flanges is highly restrained. This restraint inhibits yielding and results in somewhat more brittle behavior. Further, the stress across the beam flange connected to a wide flange column section is not uniform but rather is higher at the center of the flange and lower at the flange tips. Also, when the beam web connection is bolted rather than welded, the beam web does not participate substantially in resisting the moment; instead the beam flanges carry most of the moment. Similarly, much of the shear force at the connection is transferred through the flanges rather than through the web. These factors serve to substantially increase the stress on



(a) Connection Detail



(b) Moment-Plastic Rotation Response
of Test Specimen

Figure 1.3 Laboratory Response of W36x150 Beam with pre-Northridge Connection

the beam flange groove welds and surrounding base metal regions. Further, the weld deposit at the mid-point of the bottom flange contains "starts and stops" due to the necessity of making the flange weld through the beam web access hole. These overlapping weld deposits are both stress risers and sources of weld defects such as slag inclusions. In addition, the actual yield strength of a flexural member may exceed the nominal yield strength by a considerable amount. Since seismic design of moment frames relies on beam members reaching their plastic moment capacity, an increase in the yield strength translates to increased demands on the CJP flange weld. Several other factors have also been cited as possible contributors to the connection failures. These include adverse effects of large panel zone shear deformations, composite slab effects, strain rate effects, scale effects, and others.

Modifications to pre-Northridge WSMF connections to achieve improved seismic performance seek to reduce or eliminate some of the factors which contribute to brittle fracture mentioned above. Methods of achieving improved seismic performance are addressed in Section 2.

1.3 Repair and Modification

In the context of earthquake damage to WSMF buildings, the term *repair* is used to mean the restoration of strength, stiffness, and inelastic deformation capacity of structural elements to their original levels. Structural *modification* refers to actions taken to enhance the strength, stiffness, or deformation capacity of either damaged or undamaged structural elements, thereby improving their seismic resistance and that of the structure as a whole.

Modification typically involves substantial changes to the connection geometry that affect the manner in which the loads are transferred. In addition, structural modification may also involve the removal of existing welds and replacement with welds with improved performance characteristics.

1.4 Objective of Design Guide

A variety of approaches are possible to achieve improved seismic performance of existing welded steel moment frames. These approaches include:

- Modify the lateral force resisting system to reduce deformation demands at the connections and/or provide alternate load paths. This may be accomplished, for example, by the addition of bracing (concentric or eccentric), the addition of reinforced concrete or steel plate shear walls, or the addition of new moment resisting bays.
- Modify existing simple ("pinned") beam-to-column connections to behave as partially-restrained connections. This may be accomplished, for example, by the addition of seat angles at the connection.
- Reduce the force and deformation demands at the pre-Northridge connections through the use of measures such as base isolation, supplemental damping devices, or active control.
- Modify the existing pre-Northridge connections for improved seismic performance.

Any one or a combination of the above approaches may be appropriate for a given project. The choice of the modification strategy should carefully consider the seismic hazard at the building site, the performance goals of the modification, and of course the cost of the modification. Economic considerations include not only the cost of the structural work involved in the modification, but also the cost associated with the removal of architectural finishes and other non-structural elements to permit access to the structural frame and the subsequent restoration of these elements, as well as the costs associated with the disruption to the building function and occupants. Designers are encouraged to consult the *NEHRP Guidelines for the Seismic Rehabilitation of Buildings,* FEMA 273 (FEMA 1998) for additional guidance on a variety of issues related to the seismic rehabilitation of buildings.

Of the various approaches listed above for modification of welded steel moment frames, this Design Guide deals only with the last, i.e., methods to modify existing pre-Northridge connections for improved seismic performance. In particular, this Design Guide presents methods to significantly enhance the plastic rotation capacity, i.e., the ductility of existing connections.

There are many ways to improve the seismic performance of pre-Northridge welded moment connections and a number of possibilities are presented in *Interim Guidelines: Evaluation, Repair, Modification and Design of Steel Moment Frames, FEMA 267* (FEMA 1995) *and Advisory No. 1, FEMA 267A* (FEMA 1997).[3] Three of the most promising methods of seismic modification are presented here. There are indeed other methods which may be equally effective in improving the seismic performance of WSMF construction.

While much of the material presented in this Design Guide is consistent with *Interim Guidelines* or *Advisory No. 1,* there are several significant differences. These differences are necessitated by circumstances particular to the modification of existing buildings and by virtue of the desire to calibrate the design requirements to test data. The reader is cautioned where significant differences with either *Interim Guidelines* or *Advisory No. 1* exist.

The issue of whether or not to rehabilitate a building is not covered here. This decision is a combination of engineering and economic considerations and, until such time as modification is required by an authority having jurisdiction, the decision of whether to strengthen an existing building is left to the building owner. Studies currently in progress under the SAC Program to Reduce the Earthquake Hazards of Steel Moment-Resisting Frame Structures (Phase 2) are addressing these issues and may provide guidance in this area. Some discussion related to the need to retrofit existing steel buildings may be found in *Update on the Seismic Safety of Steel Buildings, A Guide for Policy Makers* (FEMA 1998).

If it is decided to modify an exiting WSMF building, the question arises as to whether to modify all, or only some, of the connections. This aspect too is not covered in this document as it is viewed as a decision which must be answered on a case-by-case basis and requires the benefit of a sound engineering analysis.

For a building that has already suffered some damage due to a prior earthquake, the issue of repairing that damage is of concern. Repair of existing fractured elements is covered in the *Interim Guidelines* (FEMA 1995) and is not covered here.

[3] These two reports are cited frequently herein and for brevity are referred to by *Interim Guidelines or Advisory No. 1.*

Chapter 2
ACHIEVING IMPROVED SEISMIC PERFORMANCE

The region of the connection near the face of the column may be vulnerable to fracture due to a variety of reasons, including:

- Low toughness weld metal,
- The presence of notches caused by weld defects, left in place steel backing, left in place weld tabs, and poor weld access hole geometry,
- Excessively high levels of stress in the vicinity of the beam flange groove welds and at the toe of the weld access hole, and
- Conditions of restraint which inhibit ductile deformation.

There are several approaches to minimize the potential for fracture including,

- Strengthening the connection and thereby reducing the beam flange stress,
- Limiting the beam moment at the column face, or
- Increasing the fracture resistance of welds.

Any of these basic approaches, or a combination of them, may be used. This Design Guide presents three connection modification methods: welded haunch, bolted bracket, and reduced beam section. The first two of these modification methods employ the approach of strengthening the connection and thereby forcing inelastic action to take place in the beam section away from the face of the column and the CJP flange welds. The third method seeks to limit the moment at the column face by reducing the beam section, and hence the plastic moment capacity, at some distance from the column. For those modification methods employing welding, additional steps are taken to increase the fracture resistance of the beam-to-column welds such as increasing the fracture toughness of the filler metal, reducing the size of defects, removal of steel backing and weld tabs, etc. The three modification methods covered in this Guideline are described here.

2.1 Reduced Beam Section

The reduced beam section (or RBS) technique is illustrated in Figure 2.1. As shown, the beam flange is reduced in cross section thereby weakening the beam in flexure. Various profiles have been tried for the reduced beam section as illustrated in Figure 2.2. Other profiles are also possible. The intent is to force a plastic hinge to form in the reduced section. By introducing a structural "fuse" in the reduced section, the force demand that can be transmitted

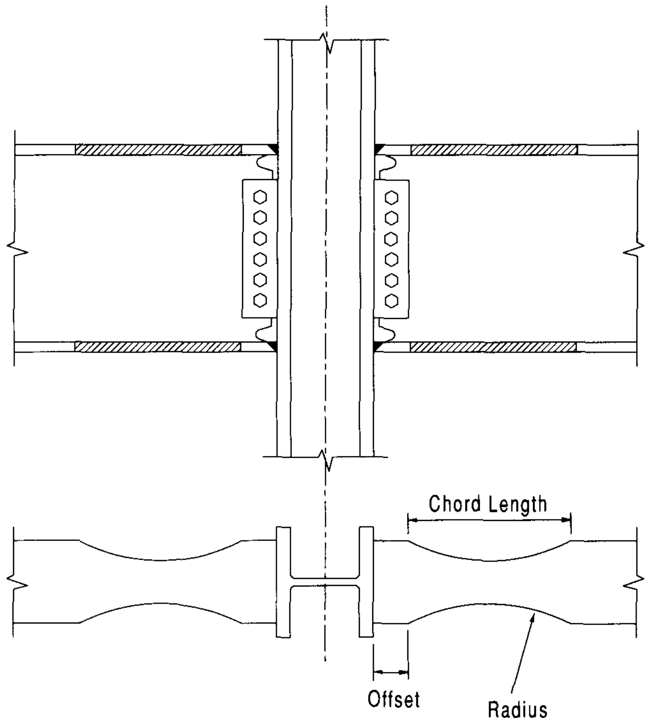


Figure 2.1 Reduced Beam Section (RBS)

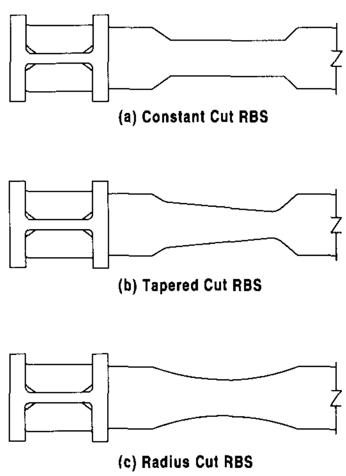


Figure 2.2 Typical Profiles of RBS Cutouts

to the CJP flange welds is also reduced. The reduction in beam strength is, in most cases, acceptable since drift requirements frequently govern moment frame design and the members are larger than needed to satisfy strength requirements. This technique has been shown to be quite promising in tests intended for new construction.

The RBS plays a role quite similar to that of connection reinforcement schemes such as cover plates, ribs, and haunches. Both the RBS and connection reinforcement move the plastic hinge away from the face of the column and reduce stress levels in the vicinity of the CJP flange welds. Connection reinforcement often requires welds that are difficult and costly to make and inspect. These problems are lessened with the RBS, which is relatively easier to construct. On the other hand, a greater degree of stress reduction can be achieved with connection reinforcement. For example, the size of haunches can be increased to achieve any desired level of stress reduction. With the RBS, on the other hand, there is a practical limit to the amount of flange material which can be removed. Consequently, there is a limit to the degree of stress reduction that can be achieved with the RBS.

The reduced beam section appears attractive for the modification of existing connections because of its relative simplicity, and because it does not increase demands on the column and panel zone. For new construction, RBS cuts are typically provided in both the top and bottom beam flanges. However, when modifying existing connections, making an RBS cut in the top flange may prove difficult due to the presence of a concrete floor slab. Consequently, in the Design Guide, design criteria are provided for modifying existing connections with the RBS cut provided in the bottom flange only.

2.2 Welded Haunch

Welding a tapered haunch to the beam bottom flange (see Figure 2.3) has been shown to be very effective for enhancing the cyclic performance of damaged moment connections (SAC 1996) or connections for new construction (Noel and Uang 1996). The cyclic performance can be further improved when haunches are welded to both top and bottom flanges of the beam (SAC 1996) although such a scheme requires the removal of the concrete floor slab in existing buildings. Reinforcing the beam with a welded haunch can be viewed as a means of increasing the section modulus of the beam at the face of the column. It will be shown in Section 6, however, that a more appropriate approach is to treat the flange of the welded haunch as a diagonal strut. This strut action drastically changes the force transfer mechanism of this type of connection.

The tapered haunch is usually cut from a structural tee or wide flange section although it could be fabricated from plate. The haunch flange is groove welded to the beam and column flanges. The haunch web is then fillet welded to the beam and column flanges (see Figure 2.3). Alternatively, using a straight haunch by connecting the haunch web to the beam bottom flange (see Figure 2.4) has been investigated for new construction (SAC 1996). However, the force transfer mechanism of the straight haunch differs from that of the tapered haunch because a direct strut action does not exist. Test results have shown that the straight haunch is still a viable solution if the stress concentration at the free end of the haunch, which tends to unzip the weld between the haunch web and beam flange, can be alleviated. In this Design Guide, only the tapered haunch is considered.

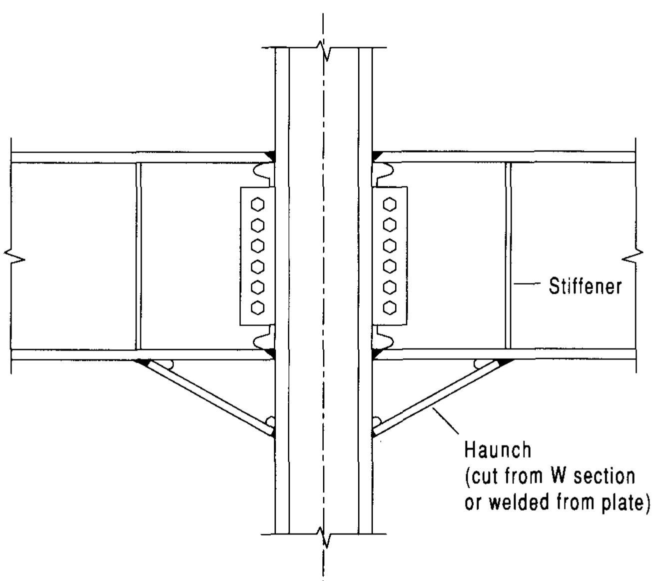


Figure 2.3 Welded Haunch

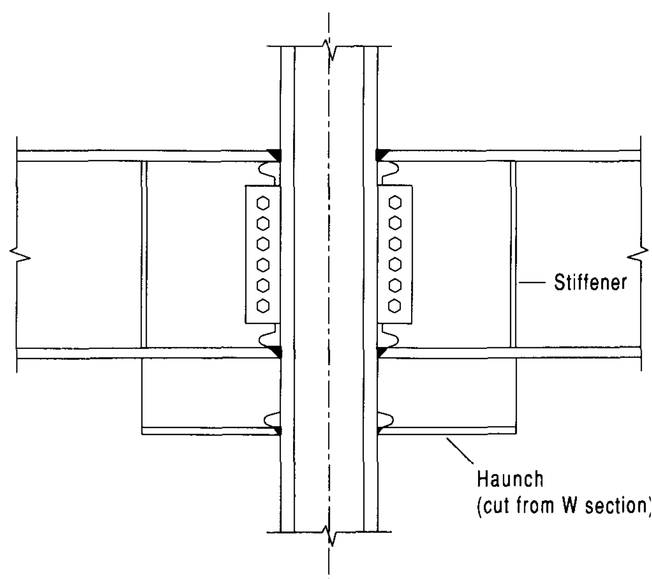


Figure 2.4 Straight Haunch

2.3 Bolted Bracket

The bolted bracket is an alternative to the welded haunch and has the added advantage that no field welding is required. Rather, high strength bolts are used to attach the bracket to both the beam and column as shown in Figure 2.5. Installation of the bolted bracket eliminates the problems associated with welding such as venting of welding fumes, supply of fresh air, and the need for fire protection.

As with the welded haunch, the bolted bracket forces inelastic action in the beam outside the reinforced region. Tests have shown this to be an effective repair and modification technique producing a rigid connection with stable hysteresis loops and high ductility (Kasai et al. 1997, 1998).

Various types of bolted bracket have been developed. The *haunch bracket* (Figure 2.5) consists of a shop-welded horizontal leg, vertical leg, and vertical stiffener. The two legs are bolted to the beam and column flanges. The *pipe bracket* (Figure 2.6) consists of pipes which are shop-welded to a horizontal plate. The plate and pipes are bolted to the beam and column flanges, respectively. The *angle bracket* (Figure 2.7) uses an angle section cut from a relatively heavy wide flange section with the flange forming the vertical leg and the web forming the horizontal leg. For light beams, hot rolled angle sections may be sufficient.

Both pipe and angle brackets have the advantage of smaller dimension compared to the haunch bracket and can therefore be embedded in the concrete floor slab. However, for heavy beam sections, it may be necessary to place a pipe or angle bracket on both sides of the beam flange which may make fabrication and erection more costly than would be the case for the haunch bracket.

When attaching the bracket to only one side of the beam flange, the use of a horizontal washer plate on the opposite side of the flange (see Figure 2.5) has been shown to enhance connection ductility. It prevents propagation of flange buckling into the flange net area that otherwise may cause early fracture of the net area. Also, the use of a thin brass plate between the bracket and beam flange has been found to be effective in preventing both noise and galling associated with interface slip.

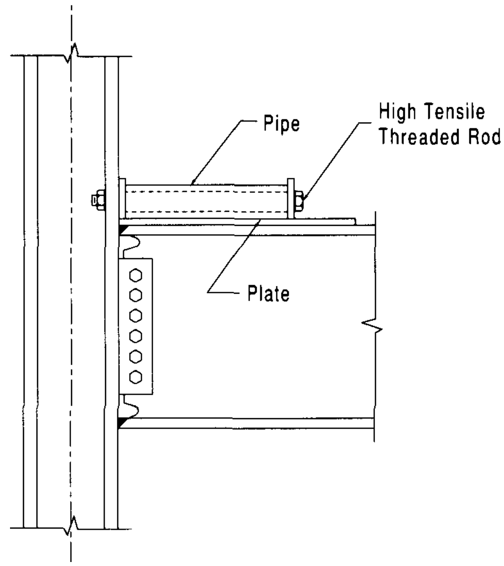


Figure 2.6 Pipe Bracket

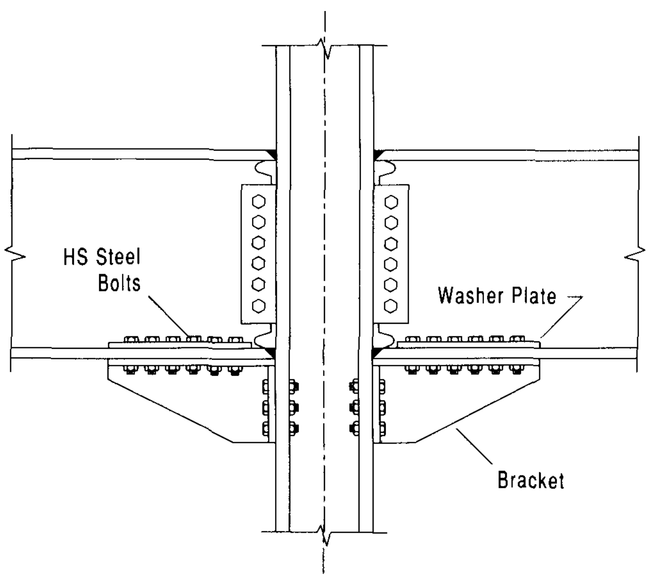


Figure 2.5 Bolted Bracket

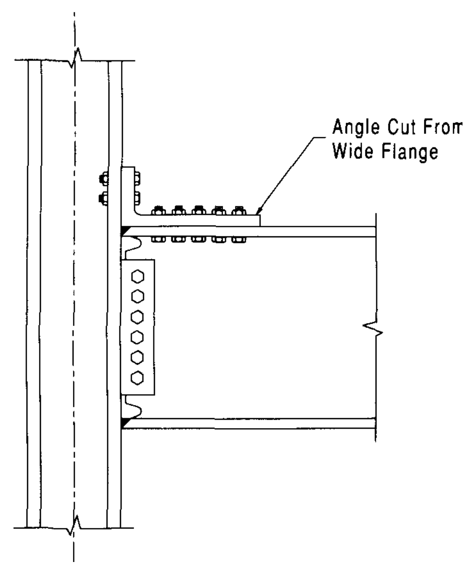


Figure 2.7 Angle Bracket

Chapter 3
EXPERIMENTAL RESULTS

Tests on full-size beam-to-column connection specimens have been conducted by a number of researchers. Experimental results that are relevant to the modification concepts addressed in this Design Guide are summarized in this section. The tests reported here were directed toward the repair and modification of pre-Northridge connections with or toward new construction. The modification of pre-Northridge moment connections differs from new construction in two significant ways:

- Existing welds are generally of low toughness E70T-4 weld metal with steel backing left in place and their removal and replacement using improved welding practices and tougher filler metal is both difficult and expensive.
- Access to the connection may be limited, especially by the presence of a concrete floor slab which may limit or preclude any modifications to the top flange.

With these limitations in mind, the National Institute of Standards and Technology (NIST) and the American Institute of Steel Construction (AISC) initiated an experimental program for the express purpose of determining the expected connection performance for various levels of connection modification. As such, initial tests were conducted on specimens that typically involved modifications only to the bottom flange. Based on successes and failures, additional remedial measures were applied until acceptable performance levels were obtained.

As already mentioned, there is a considerable amount of related research which is directed either toward the repair and modification of pre-Northridge connections or toward new construction. Tests sponsored by the SAC Joint Venture, the National Science Foundation, the steel industry and the private sector have been, and continue to be, conducted employing a variety of measures to improve the seismic performance of WSMF connections. This related research is presented in Section 3.1 followed by research results of the NIST/AISC experimental program in Section 3.2.

3.1 Related Research

A considerable amount of research has been conducted on the modification of WSMF connections to improve their seismic performance. The body of work which is relevant to the reduced beam section, welded haunch, and bolted bracket is presented here.

3.1.1 Reduced Beam Section

The majority of past research on RBS moment connections has been directed toward new construction rather than toward modification of pre-Northridge connections. Examination of data from these tests, however, provides some useful insights applicable to modification of pre-Northridge connections. As indicated in Table 3.1, a significant amount of testing has been completed over the last several years on RBS connections. On the order of thirty medium and large scale tests are summarized in this table, including a limited number of tests including a composite slab and a limited number involving dynamic loading. Examination of this data reveals that the majority of these tests were quite successful with the connections developing at least 0.03 radian plastic rotation. A few connections experienced fractures within the RBS or in the vicinity of the beam flange groove welds. Even for these cases, however, the specimens developed on the order of 0.02 radian plastic rotation and sometimes more. Consequently, the available test data for new construction suggests that the RBS connection can develop large levels of plastic rotation on a consistent and reliable basis. The RBS connection is, in fact, being employed on an increasingly common basis for new WSMF construction.

In examining the RBS data for new construction, it is important to note that most specimens, in addition to incorporating the RBS, also incorporated significant improvements in welding and in other detailing features as compared to the pre-Northridge connection. All specimens used welding electrodes which exhibit improved notch toughness as compared to the E70T-4 electrode commonly used prior to the Northridge Earthquake. The majority of specimens also incorporated improved practices with respect to steel backing and weld tabs. In most cases, bottom flange steel backing was removed, and top flange steel backing was seal welded to the column. Further, weld tabs were removed in most cases. In addition to welding related improvements, most specimens also incorporated additional detailing improvements. For example, all specimens employed continuity plates at the beam-to-column connection, although many would not have required them based on UBC requirements in force prior to the Northridge Earthquake. Many specimens incorporated additional features to further reduce stress levels at the beam flange groove welds. The majority of large scale specimens (W27 and larger beams) used welded beam

Table 3.1 Summary of Related Research Results for the Reduced Beam Section Modification								
Ref.	**Spec.**	**Beam**	**Column**	**Flange Welds**	**Web Connection**	**RBS Details and Other Flange Modifications**	**θ_P (%)**	**Comments**
[1]	YC-1	Built-up W shape d = 23.6", b_f = 11.8", t_f = 0.79", t_w = 0.47" L_b = 73" A36 steel F_{y-f} = 40 ksi F_{u-f} = 66 ksi F_{y-w} = 40 ksi F_{u-w} = 65 ksi	Built-up Box: 19.7" × 19.7" × .79" L_c = 87" A572 Gr. 50 F_y = 56 ksi F_u = 82 ksi	FCAW E70T-7 No weld tabs used	Bolted: 7-7/8" A325	Tapered cut L_1 = 2" L_{RBS} = 13.8" FR = 20%	2.1	Fracture of beam flange initiating at weld access hole
[1]	YC-2	"	"	"	"	Tapered cut L_1 = 2" L_{RBS} = 17.7" FR = 25%	2.5	Fracture of beam flange initiating at weld access hole
[1]	PC-1	"	"	"	"	Tapered cut L_1 = 4.7" L_{RBS} = 15.7" FR = 34%	3.5	Fracture of beam flange initiating at weld access hole
[1]	PC-2	"	"	"	"	Tapered cut L_1 = 4.7" L_{RBS} = 17.7" FR = 42%	4.1	Fracture of beam flange initiating at weld access hole
[1]	PC-3	"	"	"	"	Tapered cut L_1 = 4.7" L_{RBS} = 17.7" FR = 42%	3.3	Fracture of beam flange initiating at weld access hole
[2]	DBT-1A-99-176	W30×99 A572 Gr. 50 L_b = 138" F_{y-w} = 61.6 ksi F_{u-w} = 82.8 ksi	W14×176 A572 Gr. 50 L_c = 168" F_{y-w} = 55.6 ksi F_{u-w} = 70.7 ksi	FCAW E70TG-K2; backing bar removed at bottom flange	Bolted: 7-1" A325	Tapered cut L_1 = 7.5" L_{RBS} = 20.25" FR = 45%	2.7	no failure; test stopped due to limitations in test setup
[2]	DBT-1B-99-176	W30×99 A572 Gr. 50 L_b = 138" F_{y-w} = 51.5 ksi F_{u-w} = 72.1 ksi	W14×176 A572 Gr. 50 L_c = 168" F_{y-w} = 55.5 ksi F_{u-w} = 71.8 ksi	"	"	Tapered cut L_1 = 7.5" L_{RBS} = 20.25" FR = 45%	3.8	no failure; test stopped due to limitations in test setup
[2]	DBT-2A-150-257	W36×150 A572 Gr. 50 L_b = 138" F_{y-w} = 60.2 ksi F_{u-w} = 72.3 ksi	W14×257 A572 Gr. 50 L_c = 168" F_{y-w} = 59.6 ksi F_{u-w} = 75.2 ksi	"	Bolted: 9-1" A325	Tapered cut L_1 = 9" L_{RBS} = 24" FR = 45%	3.3	Fracture of beam top flange near groove weld
[2]	DBT-2B-150-257	W36×150 A572 Gr. 50 L_b = 138" F_{y-w} = 62.9 ksi F_{u-w} = 83.1 ksi	W14×257 A572 Gr. 50 L_c = 168" F_{y-w} = 64.5 ksi F_{u-w} = 83.2 ksi	"	"	Tapered cut L_1 = 9" L_{RBS} = 24" FR = 45%	1.7	Fracture of beam top flange weld; propagated to divot-type fracture of column flange

Table 3.1 (cont'd) Summary of Related Research Results for the Reduced Beam Section Modification								
Ref.	Spec.	Beam	Column	Flange Welds	Web Connection	RBS Details and Other Flange Modifications	θ_P (%)	Comments
[3,4]	ARUP-1	W36×150 A572 Gr. 50 L_b = 132" F_{y-f} = 55.5 ksi F_{u-f} = 73 ksi F_{y-w} = 62.5 ksi F_{u-w} = 77 ksi	W14×426 A572 Gr. 50 L_c = 136"	FCAW E70TG-K2 backing bar left in place w/ seal weld at top flange; backing bar removed at bottom flange	welded (heavy shear tab groove welded to column and fillet welded to beam web)	Tapered cut L_1 = 9" L_{RBS} = 24" FR = 44% top & bottom flanges reinforced with vertical ribs	3.3	Flange fracture at minimum section of RBS
[3,4]	COH-1	W27×178 A572 Gr. 50 L_b = 132" F_{y-f} = 44 ksi F_{u-f} = 62 ksi F_{y-w} = 46 ksi F_{u-w} = 62 ksi	W14×455 A572 Gr. 50 L_c = 136" F_{y-f} = 55 ksi F_{u-f} = 84 ksi F_{y-w} = 54 ksi F_{u-w} = 86 ksi	"	"	Tapered cut L_1 = 7" L_{RBS} = 20" FR = 38% top & bottom flanges reinforced with vertical ribs	3.3	"
[3,4]	COH-2	"	"	"	"	"	3.5	"
[3,4]	COH-3	W33×152 A572 Gr. 50 L_b = 132" F_{y-f} = 57.6 ksi F_{u-f} = 78.5 ksi F_{y-w} = 62 ksi F_{u-w} = 84.5 ksi	W14×455 A572 Gr. 50 L_c = 136" F_{y-f} = 55 ksi F_{u-f} = 84 ksi F_{y-w} = 54 ksi F_{u-w} = 86 ksi Beam connected to column web	"	"	Tapered cut L_1 = 9" L_{RBS} = 26" FR = 43% top & bottom flanges reinforced with vertical side plates	3.0	"
[3,4]	COH-4	"	"	"	"	"	3.7	"
[3,4]	COH-5	W33×152 A572 Gr. 50 L_b = 132" F_{y-f} = 62.8 ksi F_{u-f} = 86 ksi F_{y-w} = 69.1 ksi F_{u-w} = 93.7 ksi	"	"	"	"	1.7	"
[5,6]	DB1	W36×160 L_b = 134" F_{y-f} = 54.7 ksi F_{u-f} = 75.6 ksi F_{y-w} = 53.5 ksi F_{u-w} = 79.2 ksi	W14×426 A572 Gr. 50 L_c = 136"	FCAW E71T-8 backing bar left in place w/ seal weld at top flange; backing bar removed at bottom flange	welded (beam web groove welded to column)	Constant cut L_1 = 9" L_{RBS} = 19.5" FR = 40%	1.9	Flange fracture at RBS

Table 3.1 (cont'd)
Summary of Related Research Results for the Reduced Beam Section Modification

Ref.	Spec.	Beam	Column	Flange Welds	Web Connection	RBS Details and Other Flange Modifications	θ_P (%)	Comments
[5,6]	DB2	W36×150 L_b = 134" F_{y-f} = 41.4 ksi F_{u-f} = 58.7 ksi F_{y-w} = 47.1 ksi F_{u-w} = 61.8 ksi	W14×426 A572 Gr. 50 L_c = 136" F_{y-f} = 50 ksi F_{u-f} = 74.5 ksi F_{y-w} = 50 ksi F_{u-w} = 75 ksi	"	"	Radius cut L_1 = 9" L_{RBS} = 27" FR = 40%	2.8	Testing stopped due to limitations of test setup
[5,6]	DB3	W36×170 L_b = 134" F_{y-f} = 58 ksi F_{u-f} = 73 ksi F_{y-w} = 58.5 ksi F_{u-w} = 76.7 ksi	W14×426 A572 Gr. 50 L_c = 136"	"	"	Radius cut L_1 = 9" L_{RBS} = 27" FR = 40%	3.6	"
[5,6]	DB4	W36×194 L_b = 134" F_{y-f} = 38.5 ksi F_{u-f} = 58.6 ksi F_{y-w} = 43.6 ksi F_{u-w} = 59.8 ksi	W14×426 A572 Gr. 50 L_c = 136" F_{y-f} = 50 ksi F_{u-f} = 74.5 ksi F_{y-w} = 50 ksi F_{u-w} = 75 ksi	"	"	Radius cut L_1 = 9" L_{RBS} = 27" FR = 38%	3.5	"
[5,6]	DB5	W30×148 L_b = 134" F_{y-f} = 46.6 ksi F_{u-f} = 64.5 ksi F_{y-w} = 48.5 ksi F_{u-w} = 65.4 ksi	W14×257 A572 Gr. 50 L_c = 136" F_{y-f} = 48.7 ksi F_{u-f} = 69 ksi F_{y-w} = 49.4 ksi F_{u-w} = 66.2 ksi	"	"	Radius cut L_1 = 5" L_{RBS} = 25" FR = 38%	3.8	Testing stopped due to limitations of test setup; significant column panel zone yielding
[7]	DB1	W36×135 A36 Steel L_b = 134.5"	W14×257 with 1-5/16" thk. cover plates (cover plates welded across flanges of W14×257 to from box) A572 Gr. 50 L_c = 132"	Not Available	Not Available	Radius cut L_1 = 8" L_{RBS} = 28" FR = 40%	2.8	Testing stopped due to limitations of test setup
[7]	DB2	W36×245 A36 L_b = 134.5"	W14×398 A572 Gr. 50 L_c = 132"	FCAW E71T-8	welded (heavy welded shear tab)	Radius cut L_1 = 11" L_{RBS} = 25" FR = 40%; RBS provided in bottom flange only; top and bottom flanges provided with supplementary reinforcing plates	2.8	Testing stopped due to limitations of test setup

Table 3.1 (cont'd) Summary of Related Research Results for the Reduced Beam Section Modification								
Ref.	**Spec.**	**Beam**	**Column**	**Flange Welds**	**Web Connection**	**RBS Details and Other Flange Modifications**	θ_P (%)	**Comments**
[8]	S-1	W530×82 (Canadian Designation) d = 20.8", b_f = 8.2", t_f = 0.52", t_w = 0.37" wt. = 54 lb/ft. L_b = 142" CSA G40.41-350W steel F_{y-f} = 52.4 ksi F_{u-f} = 76.6 ksi F_{y-w} = 57.5 ksi F_{u-w} = 81 ksi	W14×120 A572 Gr. 50 L_c = 120"	FCAW E71T-8 backing bar left in place w/ seal weld at top flange; backing bar removed at bottom flange	Bolted: 5-1" A325	Radius cut L_1 = 4.7" L_{RBS} = 15.7" FR = 55%	8.3	Specimen loaded monotonically; testing stopped due to limitations of test setup
[8]	S-2A	"	"	"	"	"	3.3	Testing stopped due to limitations of test setup
[8]	SC-1	"	"	"	"	"	3.1	Composite slab included (6); testing stopped due to limitations of test setup
[8]	S-3	"	"	"	"	"	note (8)	statically applied simulated earthquake loading (7); testing stopped due to reaching end of simulated earthquake loading; no connection failure
[8]	S-4	"	"	"	"	"	note (9)	dynamically applied simulated earthquake loading (7); testing stopped due to reaching end of simulated earthquake loading; no connection failure

Table 3.1 (cont'd)
Summary of Related Research Results for the Reduced Beam Section Modification

Ref.	Spec.	Beam	Column	Flange Welds	Web Connection	RBS Details and Other Flange Modifications	θ_P (%)	Comments
[8]	SC-2	"	"	"	"	"	Note (9)	Composite slab included (6); dynamically applied simulated earthquake loading (6); testing stopped due to reaching end of simulated earthquake loading; no connection failure

Notes:

(1) All specimens are single cantilever type.
(2) All specimens are bare steel, except SC-1 and SC-2
(3) All specimens subject to quasi static cyclic loading, with ATC-24 or similar loading protocol, except S-1, S-3, S-4 and SC-2
(4) All specimens provided with continuity plates at beam-to-column connection, except Popov Specimen DB1 (Popov Specimen DB1 was provided with external flange plates welded to column).
(5) Specimens ARUP-1, COH-1 to COH-5, S-1, S-2A, S-3, S-4, SC-1 and SC-2 provided with lateral brace near loading point and an additional lateral brace near RBS; all other specimens provided with lateral brace at loading point only.
(6) Composite slab details for Specimens SC-1 and SC-2: 118" wide floor slab; 3" ribbed deck (ribs perpendicular to beam) with 2.5" concrete cover; normal wt. concrete; welded wire mesh reinforcement; 3/4" dia. shear studs spaced at 24" (one stud in every other rib); first stud located at 29" from face of column; 1" gap left between face of column and slab to minimize composite action.
(7) Specimens S-3, S-4 and SC-2 were subjected to simulated earthquake loading based on N10E horizontal component of the Llolleo record from the 1985 Chile Earthquake. For Specimen S-3, simulated loading was applied statically. For Specimen S-4 and SC-2; simulated loading was applied dynamically, and repeated three times.
(8) Specimen S-3: Connection sustained static simulated earthquake loading without failure. Maximum plastic rotation demand on specimen was approximately 2%.
(9) Specimens S-4 and SC-2: Connection sustained dynamic simulated earthquake loading without failure. Maximum plastic rotation demand on specimen was approximately 2%.
(10) Tests conducted by Plumier not included in Table. Specimens consisted of HE 260A beams (equivalent to W10x49) and HE 300B columns (equivalent to W12x79). All specimens were provided with constant cut RBS. Beams attached to columns using fillet welds on beam flanges and web, or using a bolted end plate. Details available in Refs. 9 and 10.
(11) Shaking table tests were conducted by Chen, Yeh and Chu [1] on a 0.4 scale single story moment frame with RBS connections. Frame sustained numerous earthquake records without fracture at beam-to-column connections.

Notation:

F_{y-f} = flange yield stress from coupon tests
F_{u-f} = flange ultimate stress from coupon tests
F_{y-w} = web yield stress from coupon tests
F_{u-w} = web ultimate stress from coupon tests
L_b = Length of beam, measured from load application point to face of column
L_c = Length of column
L_1 = distance from face of column to start of RBS cut
L_{RBS} = length of RBS cut
FR = Flange Reduction = (area of flange removed/original flange area) x 100(Flange Reduction reported at narrowest section of RBS)
θ_P = Maximum plastic rotation developed for at least one full cycle of loading, measured with respect to the centerline of the column

References:

[1] Chen, S.J., Yeh, C.H. and Chu, J.M, "Ductile Steel Beam-to-Column Connections for Seismic Resistance," *Journal of Structural Engineering,* Vol. 122, No. 11, November 1996, pp. 1292-1299.
[2] Iwankiw, N.R., and Carter, C., "The Dogbone: A New Idea to Chew On," *Modem Steel Construction,* April 1996.
[3] Zekioglu, A., Mozaffarian, H., and Uang, C.M., "Moment Frame Connection Development and Testing for the City of Hope National Medical Center," *Building to Last - Proceedings of Structures Congress* XV, ASCE, Portland, April 1997.
[4] Zekioglu, A., Mozaffarian, H., Chang, K.L., Uang, C.M. and Noel, S., "Designing After Northridge," *Modem Steel Construction,* March 1997.
[5] Engelhardt, M.D., Winneberger, T., Zekany, A.J. and Potyraj, T.J., "Experimental Investigation of Dogbone Moment Connections," *Proceedings; 1997 National Steel Construction Conference,* American Institute of Steel Construction, May 7-9, 1997, Chicago.
[6] Engelhardt, M.D., Winneberger, T., Zekany, A.J. and Potyraj, T.J., "The Dogbone Connection, Part II, *Modern Steel Construction,* August 1996.
[7] Popov, E.P., Yang, T.S. and Chang, S.P., "Design of Steel MRF Connections Before and After 1994 Northridge Earthquake," *International Conference on Advances in Steel Structures,* Hong Kong, December 11-14, 1996. Also to be published in: *Engineering Structures,* 20(12), 1030-1038, 1998.
[8] Tremblay, R., Tchebotarev, N. and Filiatrault, A., "Seismic Performance of RBS Connections for Steel Moment Resisting Frames: Influence of Loading Rate and Floor Slab," *Proceedings, Stessa '97,* August 4-7, 1997, Kyoto, Japan.
[9] Plumier, A., "New Idea for Safe Structures in Seismic Zones," *IABSE Symposium • Mixed Structures Including New Materials,* Brussels, 1990.
[10] Plumier, A., "The Dogbone: Back to the Future," *Engineering Journal,* American Institute of Steel Construction, Inc. 2nd Quarter 1997.

web connections rather than the more conventional bolted web. These welded beam web connections were made either by directly welding the web to the column via a complete joint penetration groove weld, or by the use of a heavy welded shear tab. Finally, in one test program (Zekioglu 1997), the RBS was supplemented by vertical reinforcing ribs at the beam-to-column connection to even further reduce stress levels.

Based on the above discussion, it seems clear that even though the beam flange cutouts are the most distinguishing feature of the RBS connection, the success of this connection in laboratory tests is also likely related to the many other welding and detailing improvements implemented in the test specimens, i.e., the use of weld metal with improved notch toughness, improved practices with respect to steel backing and weld tabs, use of continuity plates, use of welded web connections, etc. This observation has important implications for modification of pre-Northridge WSMF connections using the RBS concept. The available data suggests that simply adding an RBS cutout to the beam flanges may not, by itself, be adequate to assure significantly improved connection performance. Rather, in addition to the RBS cutout, additional connection modifications may be needed.

3.1.2 Welded Haunch

Table 3.2 summarizes the test results of eleven full-scale tapered haunch specimens that were tested after the Northridge Earthquake. Except for the last specimen which was designed for new construction, all the other specimens were tested for modification of already damaged pre-Northridge moment connection. Two of these specimens were tested dynamically. Except for three specimens that incorporated haunches at both top and bottom flanges, the other specimens had a welded haunch beneath the bottom flange only. Where a haunch was used to strengthen either the bottom or top beam flange with a fractured weld joint, the fractured flange was left disconnected.

Several schemes were used to treat the beam top flange when a haunch was added to the bottom flange only. If the top flange did not fracture during the pre-Northridge moment connection test, the existing welded joint might be left as it was if ultrasonic testing still did not show rejectable defects. A more conservative approach included reinforcing the existing top flange weld with either welded cover plate or vertical ribs. If the top flange weld fractured, the existing weld might be replaced using a notch-tough filler metal and the steel backing removed. Most of the damaged pre-Northridge specimens also experienced damage in the bolt web connection. All of the specimens reported in Table 3.2 had the beam web welded directly to the column flange.

The results in Table 3.2 show that most of the haunch specimens were able to deliver more than 0.02 radian plastic rotation. Two dynamically loaded specimens show low plastic rotation (0.014 radian) because the displacement imposed was limited due to the nature of the dynamic testing procedure. The database indicates that welded haunch is very promising for modification of pre-Northridge moment connections.

3.1.3 Bolted Bracket

Past research on bolted connections has typically addressed either gravity connections or semi-rigid moment connections. After the Northridge Earthquake, the use of a bolted bracket to create a rigid connection was studied

Table 3.2
Summary of Related Research Results for the Welded Haunch Modification

Ref.	Specimen	Beam	Column	Rehabilitation Details			θ_P (%)	Comments
				Top flange	Bottom flange	Beam Web		
[1]	UCB-RN2	W36×150 A572 Gr. 50 L_b = 135" F_{y-f} = 60.6 ksi F_{u-f} = 68.8 ksi F_{y-w} = 60.1 ksi F_{u-w} = 69.7 ksi	W14×257 A572 Gr. 50 L_c = 138"	pre-Northridge condition	Welded haunch W18×143 A572 Gr. 50 $a = 0.5d_b$ $b = 0.27d_b$ $\theta = 28°$; double bevel groove weld between new flange plate and column flange	Welded web doubler plate	1.4	Beam bottom flange fracture at end of haunch; haunch and beam stiffeners of wrong dimensions were first installed and then removed before the correct ones were installed for testing.

Table 3.2 (cont'd) Summary of Related Research Results for the Welded Haunch Modification								
Ref.	Specimen	Beam	Column	Rehabilitation Details: Top flange	Rehabilitation Details: Bottom flange	Rehabilitation Details: Beam Web	θ_P (%)	Comments
[1]	UTA-1R	W36×150 A36 L_b = 135" F_{y-f} = 42.3 ksi F_{u-f} = 61.1 ksi F_{y-w} = 47.7 ksi F_{u-w} = 63.4 ksi	W14×257 A572 Gr. 50 L_c = 136"	replacing lower one-third of E70T-4 weld by E71T-8 weld; no steel backing	Welded haunch W18×143 A572 Gr. 50 $a = 0.5d_b$ $b = 0.31d_b$ $\theta = 31°$; double bevel groove weld between new flange plate and column flange	Welded web doubler plate	1.9	Weld fracture at beam top flange
[1]	UTA-1RB	W36×150 A36 L_b = 134" F_{y-f} = 42.3 ksi F_{u-f} = 61.1 ksi F_{y-w} = 47.7 ksi F_{u-w} = 63.4 ksi	W14×257 A572 Gr. 50 L_c = 136"	E71T-8 flange weld enhanced by tapered cover plate	Same as UTA-1R	Welded web doubler plate	2.8	Severe beam local and lateral buckling; test stopped due to limitations of test setup
[1]	UTA-3R	W36×150 A36 L_b = 134" F_{y-f} = 42.3 ksi F_{u-f} = 61.1 ksi F_{y-w} = 47.7 ksi F_{u-w} = 63.4 ksi	W14×257 A572 Gr. 50 L_c = 136"	Welded haunch W18×143 A572 Gr. 50 $a = 0.5d_b$ $b = 0.31d_b$ $\theta = 31°$; E70T-4 flange weld with defect left in place	Welded haunch W18×143 $a = 0.5d_b$ $b = 0.31d_b$ $\theta = 31°$; fractured bottom flange disconnected from column	Welded web doubler plate	2.3	Severe beam local and lateral buckling; test stopped due to limitations of test set up
[1]	RFS-RN2	W30×99 A36 L_b = 134" F_{y-f} = 48.6 ksi F_{u-f} = 70.8 ksi F_{y-w} = 57.4 ksi F_{u-w} = 72.9 ksi	W14×176 A572 Gr. 50 L_c = 136"	Welded haunch W30×148 A572 Gr. 50 $a = 0.6d_b$ $b = 0.24d_b$ $\theta = 22°$; fractured beam flange disconnected from column	Welded haunch W30×148 A572 Gr. 50 $a = 0.6d_b$ $b = 0.47d_b$ $\theta = 38°$; E70T-4 beam flange weld with steel backing left in place.	Groove weld between beam web and column flange	2.8	Beam top flange fracture outside of haunch due to severe local buckling

Table 3.2 (cont'd) Summary of Related Research Results for the Welded Haunch Modification								
				Rehabilitation Details				
Ref.	Specimen	Beam	Column	Top flange	Bottom flange	Beam Web	θ_P (%)	Comments
[1]	RFS-RN3	W30×99 A36 L_b = 134" F_{y-f} = 47.2 ksi F_{u-f} = 70.4 ksi F_{y-w} = 53.4 ksi F_{u-w} = 72.1 ksi	W14×176 A572 Gr. 50 L_c = 136"	Welded haunch W24×103 A572 Gr. 50 $a = 0.6d_b$ $b = 0.24d_b$ $\theta = 22°$; E71T-8 flange weld with steel backing removed	Welded haunch W14×82 A572 Gr. 50 $a = 0.6d_b$ $b = 0.41d_b$ $\theta = 34°$; E71T-8 flange weld with steel backing removed	Groove weld between beam web and column flange	2.8	Beam top flange fracture outside of haunch due to severe local buckling
[1]	UCSD-1R	W30×99 A36 L_b = 134" F_{y-f} = 46.5 ksi F_{u-f} = 67.7 ksi F_{y-w} = 57.1 ksi F_{u-w} = 72.5 ksi	W14×176 A572 Gr. 50 L_c = 136"	Pre-Northridge condition	Welded haunch W21×93 A36 $a = 0.6d_b$ $b = 0.36d_b$ $\theta = 31°$; fractured beam flange disconnected from column	Groove weld between beam web and column flange	2.5	Beam top flange fracture at the face of column after severe beam local and lateral buckling
[1]	UCSD-3R	W30×99 A36 L_b = 134" F_{y-f} = 46.5 ksi F_{u-f} = 67.7 ksi F_{y-w} = 57.1 ksi F_{u-w} = 72.5 ksi	W14×176 A572 Gr. 50 L_c = 136"	E71T-8 weld between new flange plate and column flange; steel backing removed	Welded haunch W21×93 A36 $a = 0.6d_b$ $b = 0.36d_b$ $\theta = 31°$; E70T-4 flange weld with steel backing removed	Groove weld between beam web and column flange	4.5	Beam web fracture outside of haunch due to severe beam local and lateral buckling
[2]	UCSD-4R	W30×99 A36 L_b = 134" F_{y-f} = 46.5 ksi F_{u-f} = 67.7 ksi F_{y-w} = 57.1 ksi F_{u-w} = 72.5 ksi	W14×176 A572 Gr. 50 L_c = 136"	E70T-4 welded joint reinforced by two 3/4" thick rib plates underneath beam flange	Welded haunch W21×93 A36 $a = 0.6d_b$ $b = 0.36d_b$ $\theta = 31°$; fractured beam flange disconnected from column	Groove weld between beam web and column flange	1.4	Rib plates retained the integrity of moment connection after top flange weld fractured under dynamic loading; θ_p was limited by the imposed maximum displacement

Table 3.2 (cont'd)
Summary of Related Research Results for the Welded Haunch Modification

Ref.	Specimen	Beam	Column	Rehabilitation Details: Top flange	Rehabilitation Details: Bottom flange	Rehabilitation Details: Beam Web	θ_P (%)	Comments
[2]	UCSD-5R	W30×99 A36 L_b = 134" F_{y-f} = 46.5 ksi F_{u-f} = 67.7 ksi F_{y-w} = 57.1 ksi F_{u-w} = 72.5 ksi	W14×176 A572 Gr. 50 L_c = 136"	E71T-8 weld with steel backing removed	Welded haunch W21×93 A36 $a = 0.6d_b$ $b = 0.36d_b$ $\theta = 31°$; fractured beam flange disconnected from column	Groove weld between beam web and column flange	1.4	Low-cycle fatigue fracture of beam bottom flange outside of haunch due to local buckling in four dynamic test runs; excellent energy dissipation; θ_p limited by the imposed maximum displacement
[3]	SFCCC-8	W18×86 A36 L_b = 120" F_{y-f} = 50 ksi F_{u-f} = 72 ksi F_{y-w} = 51.5 ksi F_{u-w} = 72.5 ksi W16×89 A572 Gr.50 F_{y-f} = 47.7 ksi F_{u-f} = 66.5 ksi F_{y-w} = 47.5 ksi F_{u-w} = 66.9 ksi	W24×279 A572 Gr. 50 L_c = 150"	Top flange connected to the column through a 1-in thick cover plate	Welded built-up haunch 1-in thick flange, 3/4-in thick web $a = 1.0d_b$ $b = 0.67d_b$ $\theta = 34°$; beam flange E71T-8 weld with steel backing removed	Bolted to shear plate	3.7 3.8	Low-cycle fatigue fracture of beam flange outside of haunch due to local buckling
[4]	UTA (NSF-4)	W36×150 A36 L_b = 134" F_{y-f} = 42.6 ksi F_{u-f} = 61.7 ksi F_{y-w} = 46.5 ksi F_{u-w} = 63.0 ksi	W14×455 A572 Gr. 50 L_c = 136"	Pre-Northridge connection with steel backing removed	Welded haunch W18×143 A572 Gr. 50 $a = 0.56d_b$ $b = 0.56d_b$ $\theta = 45°$; beam flange E70T-4 weld with steel backing removed	Bolted to shear plate; supplemental beam web weld along three sides of shear tab	1.5	Weld fracture at top flange of beam

Notes:
(1) All specimens are bare steel.
(2) All specimens are one-sided moment connection

Notation:
a = haunch length
b = haunch depth
d_b = beam depth
L_b = length of beam, measured from load application to face of column
L_c = length of column
θ = angle of sloped haunch
θ_p = maximum plastic rotation developed for at least one full cycle without the strength degrading below 80% of the nominal plastic moment at the column face; θ_p computation is based on a beam span to the column Centerline.

References:

[1] SAC, "Experimental Investigations of Beam-Column Subassemblages," Report No. SAC-96-01, Parts 1 and 2, SAC Joint Venture, Sacramento, CA 1996.
[2] Uang, C.-M. and Bondad, D., "Dynamic Testing of pre-Northridge and Haunch Repaired Steel Moment Connections," Report No. SSRP 96/03, University of California, San Diego, La Jolla, CA, 1996.
[3] Noel, S. and Uang, C.-M., "Cyclic Testing of Steel Moment Connections for the San Francisco Civic Center Complex," Report No. TR-96/07, University of California, San Diego, La Jolla, CA, 1996.
[4] Engelhardt, M., *Personal Communication,* University of Texas, Austin, TX, 1997.

experimentally and analytically. A total of eight tests were performed and the results are summarized in Table 3.3.

Each test specimen was a beam-column subassemblage with a single beam attached to a column by means of a bolted bracket. Four specimens used light beams (W16X40) and column (W12x65) and the other four used heavy beams (W36X150) and columns (W14X426). Beam and column sections were of ASTM A36 steel and ASTM A572, Grade 50 steel, respectively. The bolted brackets used, both haunch brackets and pipe brackets, had configurations that allow easy installation for repair or modification of pre-Northridge connections as well as for new construction.

In five specimens, brackets were bolted to both top and bottom beam flanges which were not welded to the column, thereby simulating the connection fracture condition. The purpose was to simulate repair of both flanges or new construction. In the other three specimens, the bracket was bolted only to the bottom flange, which was not welded to the column. The purpose was to study the bolted repair of fractured bottom flange, but high toughness welds rather than pre-Northridge welds were used for the top flange to observe the connection behavior as long as possible. This test therefore differs from the NIST/AISC test that used the pre-Northridge weld for the top flange (Sec. 3.2.3).

The tests showed that bolted bracket or pipe connections are capable of providing rigid moment connections with excellent cyclic plastic rotational capacities. The stiffnesses of the tested subassemblies were essentially the same as those from theoretical calculations assuming rigid joints. The yield loads were also similar to that of the welded connection, and hysteresis was very stable without pinching, especially when close-fit holes were used for the bolts connecting the bracket and beam flange. The brackets ensured inelastic deformation occurred outside the

Table 3.3
Summary of Related Research Results for the Bolted Bracket Modification

Spec.	Beam (1)	Column (1)	Flange Welds		Rehabilitation Details		θ_P (%)	Comments
			Top flange	Bottom flange	Top flange	Bottom flange		
LK-HH-1	W16×40 F_y = 52.3 ksi L = 120"	W16×40 F_y = 52.3 ksi L = 120"	Not welded	Not welded	Bolted haunch no washer plate	Bolted haunch no washer plate	4.3 (4.6)	Flange net area fracture
LK-WH-1	W16×40 F_y = 52.3 ksi L = 120"	W12×65 F_y = 53.1 ksi L = 144"	E7018 weld, backing bar left in place	Not welded	N/A	Bolted haunch washer plate	>6.6 (>7.0)	No failure
LK-HH-2	W36×150 F_y = 38.4 ksi L = 180"	W14×426 F_y = 54.3 ksi L = 144"	Not welded	Not welded	Bolted haunch no plate	Bolted haunch washer plate	5.1 (5.4)	Flange net area fracture
LK-WH-2	W36×150 F_y = 38.4 ksi L = 180"	W14×426 F_y = 54.3 ksi L = 144"	E71T-1 weld, backing bar removed	Not welded	N/A	Bolted haunch washer plate	5.0 (5.3)	Flange net area fracture
LK-HH-3	W36×150 F_y = 38.4 ksi L = 180"	W14×426 F_y = 54.3 ksi L = 144"	Not welded	Not welded	Bolted haunch washer plate brass plate	Bolted haunch washer plate brass plate	5.9 (6.3)	Flange buckling, gross area fracture
LK-PP-1	W16×40 F_y = 52.3 ksi L = 120"	W12×65 F_y = 53.1 ksi L = 144"	Not welded	Not welded	Bolted pipe no washer plate	Bolted pipe no washer plate	5.2 (5.5)	Flange net area fracture
LK-WP-1	W16×40 F_y = 52.3 ksi L = 120"	W12×65 F_y = 53.1 ksi L = 144"	E7018 weld, backing bar left in place	Not welded	N/A	Bolted pipe washer plate brass plate	7.5 (8.0)	No failure
LK-WP-2	W36×150 F_y = 38.4 ksi L = 180"	W14×426 F_y = 54.3 ksi L = 144"	E71T weld no backing bar	Not welded	N/A	Bolted pipe washer plate brass plate	6.6 (7.0)	Flange net area fracture

Notes:
(1) Yield stress was determined from flange coupon(s).
(2) Beam plastic rotation from the face of column, ( ) beam plastic rotation from the end of bracket.
(3) Loading was ATC-24 protocol except a smaller displacement increment was used.

connection region and plastic rotation was at least 0.04 radian and typically exceeded 0.05 radian (Table 3.3). Some specimens did not fail even after 0.07 radian at which point the tests had to be terminated due to limitations in the testing apparatus. In one specimen, the beam gross section outside the connection, rather than the net section, fractured due to severe cyclic flange buckling and large plastic rotation, indicating that the connection maximized the energy dissipation capacity of the beam section.

This study also produced useful techniques to create close-fit bolt holes in the field, protect the beam flange net area from fracture, and control the noise from beam-bracket slip motion beyond the yield load.

3.2 NIST/AISC Experimental Program

The NIST/AISC testing program was designed to complement other test programs that had been completed or were in progress. In the majority of the tests conducted prior to NIST involvement, the test specimens consisted of bare steel frame sub-assemblages representing one-sided (exterior) connections. The NIST/AISC program sought to obtain data on interior, or two-sided, connections to determine if such connections perform as well as one-sided connections. Additionally, the presence of a concrete slab, whether designed to act compositely or not, tends to shift the elastic neutral axis of the beam upward, thereby increasing tensile flexural strains at the bottom beam flange weld as compared to those in a bare steel frame. To address this issue, the NIST/AISC tests included a steel deck-supported lightweight concrete slab. The concrete slab was not designed for composite action; however, shear studs designed to transfer lateral forces into the moment frame forced the slab to act compositely with the steel beam.

Beam sections used in the NIST experimental program were selected to conform to those used in the SAC Phase 1 test program. Two-sided connections, however, required larger columns than those used in the SAC tests to accommodate the unbalanced beam moments. Columns were selected so as to not require the addition of column web stiffening, commonly referred to as "doubler plates." The columns selected also did not require continuity plates as would be consistent with practice in the early 1980's. The two test specimen sizes consisted of the following beam and column sections, respectively: W30X99, W12X279 andW36xl50,W14x426.

The NIST/AISC experimental program involved the testing of 18 full-size beam-to-column connections which had been modified using the techniques described herein. One specimen was repaired and re-tested. A diagram of the test specimens and representative test apparatus is shown in Figure 3.1. The tests were conducted at the University of Texas at Austin, the University of California, San Diego, and Lehigh University's ATLSS Research Center.

Specimens were fabricated using practices which predate the 1994 Northridge Earthquake. The FCAW process was used to make the CJP flange welds and E70T-4

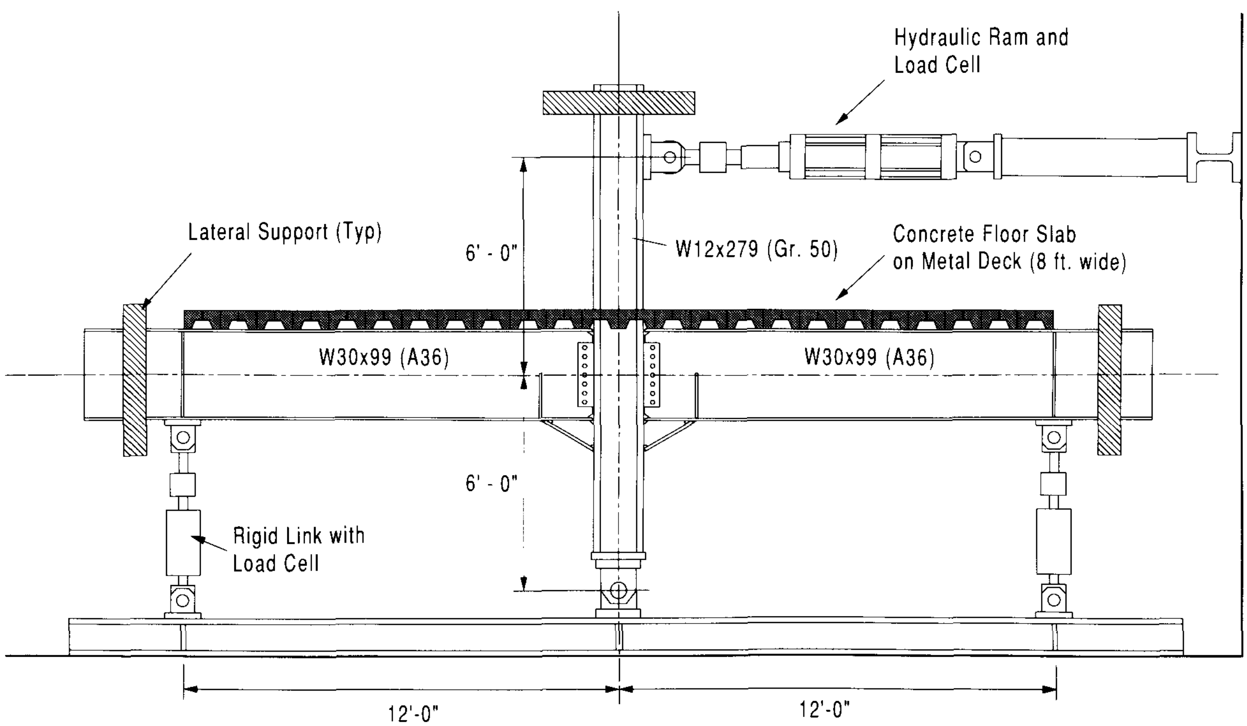


Figure 3.1 NIST/AISC Experimental Setup

electrodes were employed. The beam web was bolted to a shear tab using ASTM A325 bolts and the shear tab was welded to the column. No "return welds" were required. Also, in accordance with UBC provisions in effect in the early 1980's, neither continuity plates nor web doubler plates were required. While continuity plates would generally be required now to reflect common practice, they were omitted from this test program to better represent practice in the 1980's. The web cope was made in accordance with AWS recommended practice although inspections following the Northridge Earthquake revealed that this practice was frequently not followed. Weld tabs and weld backing were used in accordance with AWS recommended practice. The connection which was used for the NIST/AISC experimental program to represent the pre-Northridge prescriptive detail is shown in Figure 3.2.

The beam-to-column connection described above was common to all tests and indeed all specimens were made by one fabricator. The welding and bolting were completed in the upright position at the testing site using local erectors and all welds were ultrasonically inspected. The modifications were then applied as they would be in the field.

The test specimens were loaded to simulate frame response to lateral loading using hydraulic actuators (see Figure 3.1). Loads were applied in accordance with the ATC-24 (ATC 1988) loading protocol. The resulting moments were computed from measured applied forces or reaction forces and test specimen geometry. Displacements were measured and the deflection of the beam relative to the column was computed. The plastic deflection of the beam, Δ_{CL}, was obtained by subtracting the elastic beam deflection from the total beam deflection. The plastic beam rotation, θ_p, was determined from

$$\theta_p = \Delta_{CL}/L_{CL} \tag{3.1}$$

where

Δ_{CL} = plastic deflection of beam or girder, and
L_{CL} = distance between center of beam span and the centerline of the column.

The plastic beam rotation, measured in radians, is reported for all tests and is used in this document as a measure of modified connection performance. Calculation of standard uncertainty (per NIST policy) is not performed since uncertainties in material characterization are generally within 5% and are much greater than uncertainties associated with load and displacement measurements. In determining the plastic rotation capacity, the Acceptance Criteria in FEMA 267 (1995) was adopted; the Acceptance Criteria require that θ_p be the maximum plastic

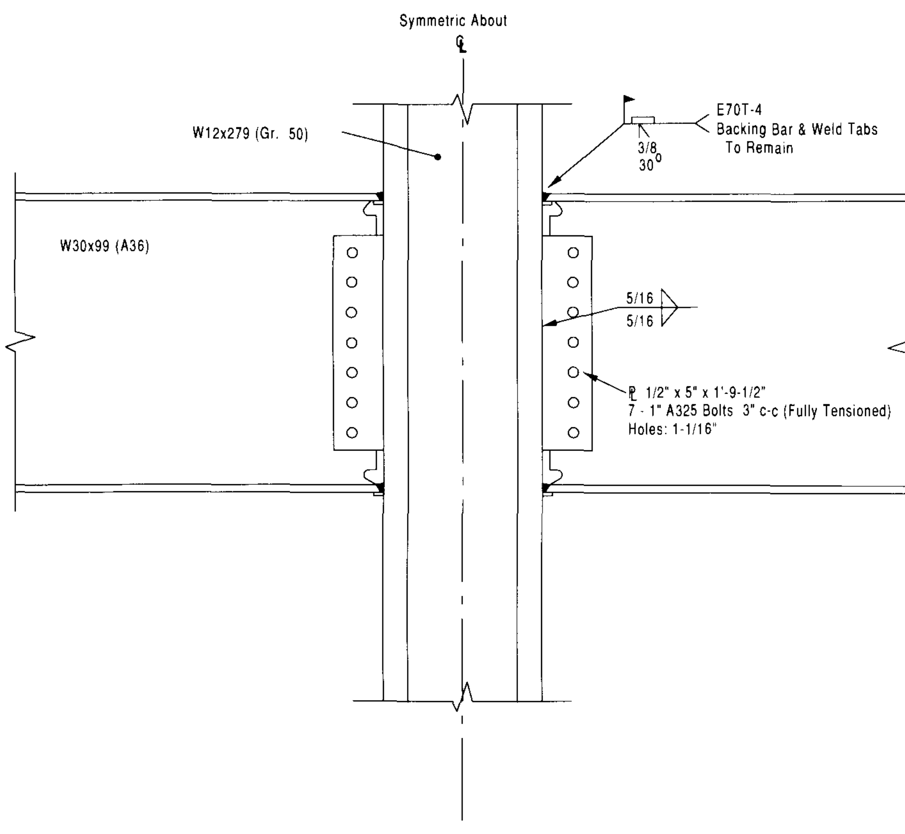


Figure 3.2 NIST/AISC Test Specimen Details

rotation developed for at least one full cycle of loading, but the beam flexural strength cannot degrade below 80% of its nominal value.

When the pre-Northridge moment connection that exhibits brittle fracture behavior (see Figure 1.3) is modified by the schemes proposed in this Design Guide, a plastic rotation capacity of at least 0.02 radian generally can be achieved. For example, Figure 3.3 shows the typical response of a welded haunch specimen with composite slab (see Figure 4.2 for the test specimen with W36X150 beams). The plastic rotation capacity was 0.028 radian. Similarly, Figure 3.4 shows that the plastic rotation capacity of a pre-Northridge moment connection with the RBS modification was 0.025 radian.

3.2.1 Reduced Beam Section

Table 3.4 summarizes tests in which pre-Northridge connections were modified with an RBS. This data supports the observation made above, i.e., the addition of the beam flange cutout, by itself, is not adequate for significantly improved connection performance. The minimum modification used in these tests was the addition of an RBS cutout in the beam bottom flange, and removal of steel backing at the beam flange groove welds. For these cases, the existing low toughness E70T-4 weld metal was left in place, no continuity plates were added, and no modifications were made to the existing bolted web connection. Tests on these connections showed poor performance. In

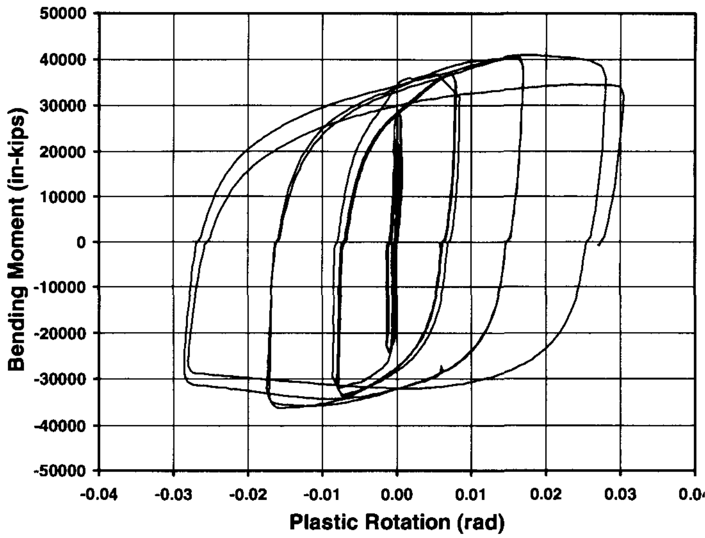


Figure 3.3 Moment-Plastic Rotation Response of a pre-Northridge Moment Connection with Welded Haunch Modification

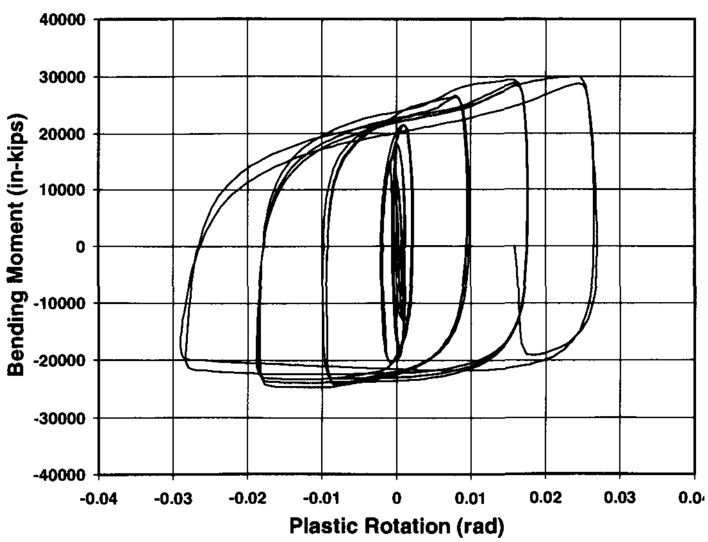


Figure 3.4 Moment-Plastic Rotation Response of a pre-Northridge Moment Connection with RBS Modification

Table 3.4
Summary of NIST/AISC Research Results for the Reduced Beam Section Modification

Specimen	Beams (1)	Column	Composite or Bare Steel (2)	Flange Welds: Top flange	Flange Welds: Bot. flange	RBS Details	θ_P (%)	Comments
UT-RBS-1	W30×99 A36 L_b = 136 F_{y-f} = 49 ksi F_{u-f} = 64 ksi F_{y-w} = 50 ksi F_{u-w} = 65 ksi	W12×279 A572 Gr. 50 L_c = 144" F_{y-f} = 48.8 ksi F_{u-f} = 66.5 ksi F_{y-w} = 56.6 ksi F_{u-w} = 71.7 ksi	Bare Steel	FCAW E70T-4 backing bar and weld tabs left in place	FCAW E70T-4 backing bar and weld tabs removed	bottom flange: radius cut L_1 = 5" L_{RBS} = 20" FR = 50%; No RBS at top flange	Beams 1 and 2: 0.6	Beams 1 and 2: fracture at bottom flange weld
UT-RBS-2	"	"	Composite	FCAW First welded with E70T-4; E70T-4 weld completely removed; Re-weld with E71T-8; weld tabs removed; backing bar left in place with seal weld to column	FCAW First welded with E70T-4; E70T-4 weld completely removed; Re-weld with E71T-8; backing bar and weld tabs removed	"	Beams 1 and 2: 2.1	Beams 1 and 2: fracture along "k-line" at bottom flange of beam causing separation of beam flange and beam web, followed by fracture of bottom flange weld
UCSD - RBS-1	W36×150 A36 steel L_b = 135" F_{y-f} = 49 ksi F_{y-w} = 47.5 ksi	W14×426 A572 Gr. 50 L_c = 144"	Bare Steel	FCAW E70T-4 backing bar and weld tabs left in place	FCAW E70T-4 backing bar and weld tabs removed	bottom flange: radius cut L_1 = 6" L_{RBS} = 27" FR = 50%; No RBS at top flange	Beam1: 0 Beam2: 1.2	Beams 1 and 2: fracture at top flange weld
UCSD - RBS-2	"	"	Composite	"	"	"	Beams 1 and 2: 1.0	Beam 1: test stopped after fracture at Beam 2 Beam 2: fracture at top flange weld
UCSD - RBS-2R (6)	"	"	"	Note (6)	Note (6)	"	Beams 1 and 2: 0.8	Beams 1 and 2: fracture at top flange weld
UCSD - RBS-3	"	"	Bare Steel	FCAW E71T-8 (7) remove backing bar and weld tabs	FCAW E71T-8 (7) remove backing bar and weld tabs	"	Beam 1: 2.9 Beam 2: 1.0	Beam 1: fracture along "k-line" of beam causing separation of beam flange and beam web followed by buckling of beam bottom flange; Beam 2: Testing stopped due to problem with test setup

Table 3.4 (cont'd) Summary of NIST/AISC Research Results for the Reduced Beam Section Modification								
Specimen	Beams (1)	Column	Composite or Bare Steel (2)	Flange Welds: Top flange	Flange Welds: Bot. flange	RBS Details	θ_P (%)	Comments
UCSD - RBS-4	"	"	Composite	"	"	"	Beams 1 and 2: 3.0	Beams 1 and 2: fracture along "k-line" of beam causing separation of beam flange and beam web followed by buckling of beam bottom flange

Notes:

(1) All specimens are two-sided.

(2) Composite slab details: 8 ft. wide floor slab; 3" ribbed metal deck (ribs perpendicular to beam) with 3.25" concrete cover; lightweight concrete with nominal f'_c = 4000 psi; welded wire mesh reinforcement; 3/4" dia. shear studs spaced nominally at 12" (one stud per rib)

(3) All specimens provided with a bolted beam web connection
W30X99 beams: 7-1" A325 bolts
W36x150 beams: 9-1" A325 bolts

(4) No specimens were provided with continuity plates.

(5) For all specimens, lateral bracing was provided near the beam ends only; no additional lateral bracing was provided at RBS for any specimen

(6) Specimen UCSD-RBS-2R was a repaired version of UCSD-RBS-2. Description of Repairs:
Fractured top flange weld of Beam 2 was removed, and rewelded with E70T-4; backing bar and weld tabs were removed;
Backing bar and weld tabs were removed from the unfractured E70T-4 top flange weld of Beam 1, and from unfractured bottom flange welds for both beams. Therefore, at completion of repairs, top and bottom flange groove welds for both beams consisted of E70T-4 weld metal, with backing bars and welds tabs removed.

(7) Specimens UCSD-RBS-3 and UCSD-RBS-4: Prior to welding flanges with E71T-8, a small portion of the column flange was removed by carbon air arc gouge, and then "buttered" with weld metal. This was intended to simulate heat effects on the column flange that would have occurred if the groove weld was first made with E70T-4, followed by removal of the E70T-4 weld metal.

Notation:

F_{y-f} = flange yield stress from coupon tests
F_{u-f} = flange ultimate stress from coupon tests
F_{y-w} = web yield stress from coupon tests
F_{u-w} = web ultimate stress from coupon tests
L_b = Length of beam, measured from load application point to face of column
L_c = Length of column
L_1 = distance from face of column to start of RBS cut
L_{RBS} = length of RBS cut
FR = Flange Reduction = (area of flange removed/original flange area) x100; (Flange Reduction reported at narrowest section of RBS)
θ_P = Maximum plastic rotation developed for at least one full cycle of loading, measured with respect to the centerline of the column

all cases, the existing low toughness beam flange groove welds fractured at low levels of plastic rotation. Apparently, the degree of stress reduction provided by the addition of a bottom flange RBS was inadequate to prevent brittle fracture of the existing low toughness welds. Further measures were required to significantly improve performance. Better performance was achieved by not only providing a flange cutout, but also by replacing the existing top and bottom beam flange groove welds with a higher toughness weld metal.

3.2.2 *Welded Haunch*

Table 3.5 summarizes tests in which pre-Northridge connections were modified with a welded haunch. For both sets of member sizes tested, the test data shows that, when the beam top flange groove welded joint was left in its pre-Northridge condition, the welded haunch modification outperformed the RBS modification. Of the three sets of bare steel specimens tested, five beams experienced weld fracture at the top flange. Two-thirds of the beams were able to experience at least one complete cycle at a story drift ratio of 2.5%. When the concrete slab was present, none of the beams experienced weld fracture. Table 3.5 shows that the plastic rotation capacity of six beams varied from 0.028 radian to 0.031 radian, more than adequate for modification purposes.

For welded haunch specimens, the yield length of the beam flanges was also significantly longer than that observed from the RBS specimens, the most significant difference being in the top flange. While the top flange yield zone of the RBS specimens was confined to a limited length next to the column face, the corresponding yield zone for the welded haunch specimens spread over a much longer distance. This desirable behavior is explained by a theory presented in Chapter 6.

Table 3.5
Summary of MIST/AISC Research Results for the Welded Haunch Modification

Specimen	Beam	Column	Rehabilitation Details			Slab	θ_P (%)	Comments
			Top flange	Bottom flange	Beam Web			
UCSD-1 (NIST-2)	W36×150 A36 L_b = 144" F_{y-f} = 49 ksi F_{u-f} = 69 ksi F_{y-w} = 47.5 ksi F_{u-w} = 65.6 ksi	W14×426 A572 Gr. 50 L_c = 144"	pre-Northridge condition	Welded haunch W18×143 A572 Gr. 50 $a = 0.5d_b$ $b = 0.31d_b$ $\theta = 31°$; beam flange E70T-4 weld with steel backing removed	Bolted to shear plate	No	0.8 2.4	Weld fracture at top flange of both beams
UCSD-1 (NIST-2C)	W36×150 A36 L_b = 144" F_{y-f} = 49 ksi F_{u-f} = 69 ksi F_{y-w} = 47.5 ksi F_{u-w} = 65.6 ksi	W14×426 A572 Gr. 50 L_c = 144"	pre-Northridge condition	Welded haunch W18×143 A572 Gr. 50 $a = 0.5d_b$ $b = 0.31d_b$ $\theta = 31°$; beam flange E70T-4 weld with steel backing removed	Bolted to shear plate	Yes	3.0 3.0	No weld fracture; test stopped after the beams experienced significant local buckling
UT-1 (HAUNCH 1)	W30×99 A36 L_b = 144" F_{y-f} = 48.5 ksi F_{u-f} = 65.0 ksi F_{y-w} = 50.0 ksi F_{u-w} = 65.9 ksi	W12×279 A572 Gr. 50 L_c = 144"	pre-Northridge condition	Welded haunch W21×93 A572 Gr. 50 $a = 0.6d_b$ $b = 0.36d_b$ $\theta = 31°$; beam flange E70T-4 weld with steel backing	Bolted to shear plate	No	1.5 2.7	Weld fracture at top flange of one beam
UT-2 (HAUNCH 2)	W30×99 A36 L_b = 144" F_{y-f} = 48.5 ksi F_{u-f} = 65.0 ksi F_{y-w} = 50.0 ksi F_{u-w} = 65.9 ksi	W12×279 A572 Gr. 50 L_c = 144"	pre-Northridge condition	Welded haunch W21×93 A572 Gr. 50 $a = 0.6d_b$ $b = 0.36d_b$ $\theta = 31°$; beam flange E70T-4 weld with steel backing	Bolted to shear plate	Yes	3.0 3.0	No weld fracture; test stopped after the beams experienced significant local buckling
UT-3 (HAUNCH 3)	W30×99 A36 L_b = 144" F_{y-f} = 49.6 ksi F_{u-f} = 65.5 ksi F_{y-w} = 52.3 ksi F_{u-w} = 67.7 ksi	W12×279 A572 Gr. 50 L_c = 144"	pre-Northridge condition	Welded haunch W21×93 A572 Gr. 50 $a = 0.6d_b$ $b = 0.36d_b$ $\theta = 31°$; beam flange E70T-4 weld with steel backing	Bolted to shear plate	No	2.5 1.3	weld fracture at top flange of both beams

Table 3.5 (cont'd) Summary of NIST/AISC Research Results for the Welded Haunch Modification								
			Rehabilitation Details					
Specimen	Beam	Column	Top flange	Bottom flange	Beam Web	Slab	θ_P (%)	Comments
UT-4 (HAUNCH 4)	W30×99 A36 L_b = 144" F_{y-f} = 49.9 ksi F_{u-f} = 65.1 ksi F_{y-w} = 50.8 ksi F_{u-w} = 66.1 ksi	W12×279 A572 Gr. 50 L_c = 144"	pre-Northridge condition	Welded haunch W21×93 A572 Gr. 50 $a = 0.6d_b$ $b = 0.36d_b$ $\theta = 31°$; beam flange E70T-4 weld with steel backing	Bolted to shear plate	Yes	3.1 3.1	No weld fracture; test stopped after the beams experienced significant local buckling

Notes:

(1) All specimens are two-sided moment connection.
(2) All specimens subject to quasi static cyclic loading, with ATC-24 loading protocol, except UCSD-4R and UCSD-5R.
(3) All specimens are laterally braced near the loading point.
(4) E71T-8 electrode was used for welding the haunch to the beam.

Notation:

a = haunch length
b = haunch depth
d_b = beam depth
L_b = length of beam, measured from load application to face of column
L_c = length of column
θ = angle of sloped haunch
θ_p = maximum plastic rotation developed for at least one full cycle without the strength degrading below 80% of the nominal plastic moment at the column face; θ_p computation is based on a beam span to the column centerline.

Table 3.6 Summary of NIST/AISC Research Results for the Bolted Bracket Modification									
			Flange Welds		Rehabilitation Details				
Spec.	Beam (1)	Column (1)	Top flange	Bottom flange	Top flange	Bottom flange	Slab	θ_P (%)	Comments
LU-1	W36×150 F_y = 49 ksi L = 144"	W14×426 F_y = 64.3 ksi L = 144"	SS-FCAW E70T-4 elect. Steel backing left in place	Not welded	N/A	Bolted haunch washer plate brass plate	No	0.2 (0.3) 0.7 (0.9)	Top flange welds fractured during displ. cycles at Δ_y and $2\Delta_y$
LU-2	W30×99 F_y = 56 ksi L = 144"	W12×279 F_y = 51 ksi L = 144"	SS-FCAW E70T-4 elect. Steel backing left in place	Not welded	N/A	Bolted haunch washer plate brass plate	No	0.5 (0.6) 1.4 (1.6)	Top flange welds fractured during displ. cycles at $3\Delta_y$
LU-3	W36×150 F_y = 49 ksi L = 144"	W14×426 F_y = 64.3 ksi L = 144"	SS-FCAW E70T-4 elect. Steel backing left in place	Not welded	N/A	Bolted haunch washer plate brass plate	Yes	0.2 (0.3) 0.7 (0.9)	Top flange welds fractured during displ. cycles at $2\Delta_y$
LU-4	W30×99 F_y = 56 ksi L = 144"	W16×40 F_y = 52.3 ksi L = 144"	SS-FCAW E70T-4 elect. Steel backing left in place	Not welded	N/A	Bolted haunch washer plate brass plate	Yes	0.2 (0.23) 0.2 (0.23)	Top flange welds fractured during displ. cycles at $2\Delta_y$ and $3\Delta_y$

Table 3.6 (cont'd) Summary of NIST/AISC Research Results for the Bolted Bracket Modification									
			Flange Welds		Rehabilitation Details				
Spec.	Beam (1)	Column (1)	Top flange	Bottom flange	Top flange	Bottom flange	Slab	θ_P (%)	Comments
LU-5	W36×150 F_y = 49 ksi L = 144"	W14×426 F_y = 69.3 ksi L = 144"	SS-FCAW (3) E70T-4 elect. Steel backing left in place	Not welded	Bolted Double angles trimmed from W36×256, Gr.50 Triangular stiffeners	Bolted haunch washer plate brass plate	No	5.0 (6.2) 5.0 (6.2)	Net area failure during displ. cycles at $5\Delta_y$
LU-6	W30×99 F_y = 51 ksi L = 144"	W12×279 F_y = 51 ksi L = 144"	SS-FCAW E70T-4 elect. Steel backing left in place	Not welded	Bolted Double angles, trimmed from W36×256, Gr.50	Bolted haunch washer plate brass plate	No	5.6 (6.5) 5.6 (6.5)	Top flange gross area failure displ. cycles at $6\Delta_y$

Notes:
(1) Yield stress was determined from mill report, coupon tests will be done later.
(2) Beam plastic rotation from the face of column, () beam plastic rotation from the end of bracket.
(3) UT indicated weld defects in both top flange welds.
(4) Loading was ATC-24 protocol.

3.2.3 Bolted Bracket

Table 3.6 summarizes the NIST/AISC tests in which pre-Northridge connections were modified using bolted brackets. For specimens LU-1 to LU-4 using either W30 or W36 beams, with or without a concrete slab, the beam bottom flange was modified by bolting the haunch bracket while the top flange pre-Northridge weld was not modified. These four specimens showed poor performance developing early fracture of the top flange weld. In contrast, previous tests of similar connections having high toughness weld at the top flange (see Section 3.1.3) consistently showed excellent performance without fracture of the weld.

Based on these four tests, it was decided for the remaining NIST/AISC tests to modify not only the bottom flange but also the top flange connections. For specimens LU-5 and LU-6, the low toughness weld at the top flange was not replaced. Rather, a stiff double angle was bolted to the beam top flange and column face for the purpose of protecting the top flange weld. For specimen LU-5, ultrasonic testing indicated weld defects in the top flanges of both of the W36 beams. Although the weld did not meet AWS standards, the defects were not repaired since welds that survived during the Northridge event were found to have small cracks in many instances.

Both specimens LU-5 and LU-6 performed excellently, exhibiting more than 0.05 radian plastic rotation, and did not show any evidence of fracture of the top flange pre-Northridge weld. Strain gage readings at the top flange welds indicated excellent stress control by the addition of the angle bracket. The angle bracket creates an additional stress path and the bolt holes in the beam flange act as a "fuse" yielding at a relatively low load and limiting the tension force in the weld at column face.

If top flange weld fracture had occurred in specimens LU-5 and LU-6, an impact load would have acted on the brackets and bolts due to the sudden shift of the flange tension force from the weld to the bracket. To examine this effect, a full-size test was conducted on a specimen similar to LU-5. In contrast, however, a relatively small single angle bracket was used to reinforce the top flange pre-Northridge weld. Fracture of the weld occurred because the bracket, which was relatively flexible, shared only a small portion of the flange tension force. The impact force did not damage either the bracket or bolts. The bottom flange weld reinforced by a much stiffer haunch bracket did not fracture.

These results as well as the results of tests LU-5 and LU-6 and finite element analyses, suggest that a strong bracket can prevent weld fracture since it can share a significant portion of the flange tension force, thereby reducing the weld stress considerably. Further, the impact due to a sudden transfer of force to the bolted device caused by a weld fracture should not have a detrimental effect on the bolts and bracket. This would be especially true when the bracket and bolts are stronger than the single angle bracket tested.

Chapter 4
DESIGN BASIS FOR CONNECTION MODIFICATION

Building frames designed in accordance with the UBC and NEHRP Recommended Provisions are intended to develop inelastic flexural or shear deformations as a means of dissipating earthquake energy. At large inelastic rotational strains, flexural behavior may be approximated by introducing the concept of plastic hinges. The prescriptive connection contained in the UBC and NEHRP Recommended Provisions (see Section 1.1) was based on the assumption that plastic hinges would form at the column faces and that material was sufficiently ductile to accommodate the large inelastic strains. The failure of many welded connections in the Northridge earthquake by brittle fracture has demonstrated that the prescribed connection is not capable of reliably providing the necessary ductility. Thus, in order to achieve improved and more reliable connection performance, moment connections should be modified so as to move the plastic hinge away from the column face. This may be accomplished either by strengthening the connection or by weakening the beam at some distance from the face of the column. The resulting frame performance is illustrated in Figure 4.1. Care must be taken to insure that, when connections are strengthened, the strong column-weak beam design requirement is still satisfied.

Connections which are modified using procedures described in this Design Guide should experience fewer brittle failures than connections which are not modified. Still, the formation of a plastic hinge, which may be accompanied by local buckling, constitutes damage which may require repair following a severe earthquake. The performance of a building modified as described herein should be significantly improved and the safety of the building occupants thereby increased as the potential for collapse is reduced. Further, in an earthquake of the magnitude of the Northridge event, it is anticipated that the need for costly repairs would be minimized.

In this section, procedures will be developed to 1) determine the expected yield strength of the connection components, 2) compute the beam moment and shear necessary for proportioning the structural modification, and 3) insure that the strong column-weak beam design requirement is satisfied. Lastly, the desired modified connection rotation capacity is discussed. The concepts set forth in this section are common to the various modification methods described in the following sections.

The connection modification procedures presented in this Design Guide are based on the experiments described in Section 3.2. These experiments were conducted on specimens constructed with W30×99 and W36×150 beams. Due to potential scale effects on the behavior of steel moment connections, caution is required when extrapolating these design procedures to sections that are substantially deeper or heavier than those tested.

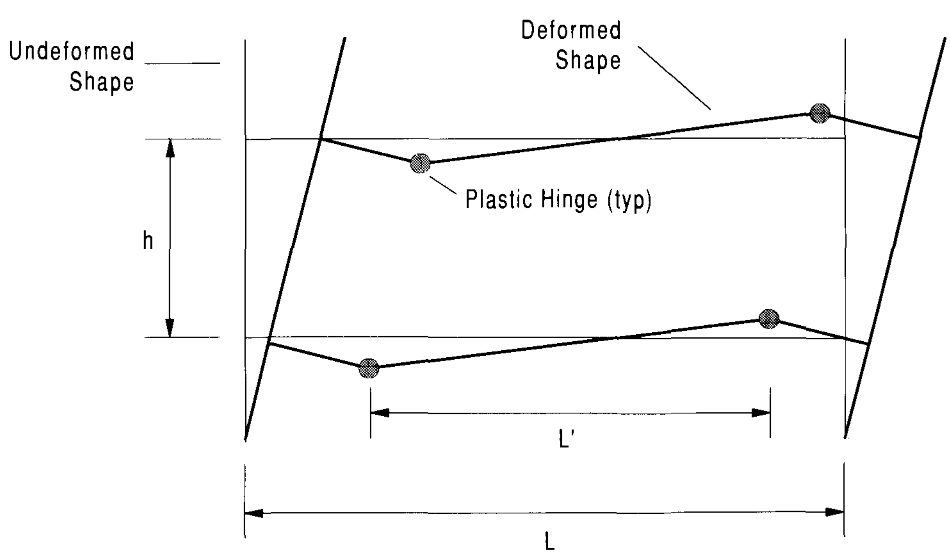


Figure 4.1 Idealized Plastic Frame Behavior

Suggested limits on the extrapolation of test results to larger members are provided in Appendix S of the AISC Seismic Provisions for Structural Steel Buildings (AISC 1997).

4.1 Material Strength

For the design of any connection modification, it is necessary to have an estimate of the yield strength of the connected members. Estimates may be obtained from compiled statistical data as presented in Table 4.1, from Certified Mill Test Reports (CMTRs) for the steel used in the construction, or from tensile tests of material removed from the structural frame to be modified. The value of flange yield strength obtained as described here and used in design calculations to follow is termed the *expected yield strength.* The AISC Seismic Provisions (AISC 1997) define the expected yield strength, F_{ye} as

$$F_{ye} = R_y F_y \tag{4.1}$$

where

R_y = a multiplier that accounts for material overstrength, and
F_y = minimum specified yield strength.

The material overstrength factor, R_y, may be determined per the AISC *Seismic Provisions for Structural Steel Buildings* as modified herein (see Table 4.1). The AISC Seismic Provisions recommend that R_y be taken as 1.5 for ASTM A36 steel. The "overstrength factor" of 1.5 reflects the distribution of yield strength of A36 steel wide flange sections in current production and the practice of multi-grade certification, which is becoming more common. This design guide, however, addresses the modification of existing buildings constructed prior to the 1994 Northridge earthquake. Prior to 1994, only relatively light sections were produced as multi-grade, sections not typically found in WSMF construction. So the main issue is one of estimating the expected dynamic flange yield strength of ASTM A36 steel.

Data from the 1992 production year (Frank 1995) shows a wide variation in the yield point of A36 steel among the various producers. The mean yield point for all producers is reported to be 49 ksi. To account for the fact that mill tests in 1992 were conducted on samples taken from the web, this value should be multiplied by 0.95, giving a flange yield point of roughly 47 ksi. No adjustments are made for the rapid testing speeds often employed by the mills (Galambos and Ravindra 1978) since the resulting higher loading rate is thought to approximate the dynamic conditions experienced in earthquake loading. Thus, the overstrength factor corresponding to this estimated yield strength is R_y = 47/36 ~ 1.3.

Table 4.1
Material Overstrength Factor, R_y, for Steels Produced Prior to 1994

ASTM Steel Grade	Rolled Shapes and Bars	Plates
A36	1.3	1.1
All other grades	1.1	1.1

Yield strength values reported on CMTRs provide only approximate estimates of actual member yield strengths and care should be exercised in the interpretation of such values. Mills routinely test tension specimens at a high loading rate and report the upper yield point, and, prior to 1997, tests were conducted on specimens removed from the web. These factors combine to produce yield strength values on the CMTR that may exceed the actual flange material dynamic yield strength.

Finally, R_y may be determined by testing conducted in accordance with requirements for the specified grade of steel. It is preferable to determine F_{ye} from material that is removed from the beam flanges. However, in some cases, it may be necessary to test material that is removed from the web which normally results in values that are on the order of 5 percent higher than those obtained from flange material (Galambos and Ravindra 1978). Thus, yield strength values obtained from the web should be multiplied by 0.95. In all cases, sufficient samples should be taken to produce meaningful results. Further, the user is cautioned not to reduce significantly the expected yield strength on the basis of a few tests as this may lead to an unconservative design.

4.2 Critical Plastic Section

For each of the three connection modifications described in this Design Guide, yielding of the beams is anticipated to occur in a region just beyond the beam-to-column connections. For the welded haunch or bolted bracket, yielding occurs in the region of the beam near the end of the haunch or bracket. In the case of the RBS modification, yielding is concentrated within the reduced section of the beam. In each of these cases, the yielded region of the beam serves as a fuse, limiting the moment and shear that can be transferred to the beam-to-column connection. That is, the yielded region of the beam controls the maximum force that can be transmitted from the beam to the CJP groove welds and other connection elements.

Design of a connection modification requires estimating the maximum moment that can be generated within the yielded region of the beam. This calculation must consider realistic estimates of beam yield stress (Section 4.1) and realistic estimates of the maximum strain hardening that may occur at large levels of plastic rotation. The anticipated level of strain hardening can be estimated from experimental data. That is, the maximum strain hardening which occurred within the yielded region of the beams can be measured in experiments, and these values can be used to estimate strain hardening factors to be used in design. In this Design Guide, strain hardening factors were determined from the NIST/AISC experimental program, and from other experiments on welded haunches, bolted brackets, and RBS type connections.

The yielded region of the beam is often referred to as a *plastic hinge.* For calculation purposes, the plastic hinge is typically treated as a single point along the length of the beam, as illustrated in Figure 4.1. In reality of course, yielding extends over a finite length of the beam. Choosing a single location along the yielded region of the beam to represent a concentrated plastic hinge is therefore subject to judgment and may pose some difficulty. Yield patterns observed in the NIST/AISC experimental program illustrate the difficulty in locating a concentrated plastic hinge, because the location and extent of flange yielding are not the same at the top and bottom flanges. Consider the welded haunch modification where the haunch is added to the bottom flange only. Figure 4.2 shows that the yielded length of the bottom flange extends outward from the haunch tip and is shorter than the yielded length of the top flange, which extends closer to the column. Thus, choosing a single point to represent a concentrated plastic hinge is somewhat arbitrary. Similar observations can be made for the bolted bracket and RBS modifications.

In this Design Guide, in order to avoid potential confusion associated with a point hinge concept, it was decided to define a convenient *critical plastic section* for each connection modification and to calibrate computed and observed strength on this basis. Table 4.2 gives the location of the critical plastic section for each modification and Figure 4.3 further illustrates the notion for clarity. For each connection modification, the critical plastic section is the point along the length of the beam where the ratio of beam flexural strength to applied moment is at or near a minimum. Thus, the critical plastic section, in a general sense, may be viewed as the cross-section within the yielded region of the beam which might be anticipated to experience the largest inelastic strains. It should be emphasized that the critical plastic section is different from the plastic hinge location recommended in *Advisory No. 1* (FEMA 1996).

Strain hardening factors for design of the connection modifications in this Design Guide have been calibrated to the critical plastic sections listed in Table 4.2. That is, the maximum strain hardening which occurred in experiments was computed at these sections. The location of the critical plastic section is of course somewhat arbitrary.

Table 4.2
Location of Critical Plastic Sections for Modified Connections

Modification	Critical Plastic Section
RBS	Centerline of RBS
Welded haunch	Tip of haunch
Bolted bracket	Tip of bracket

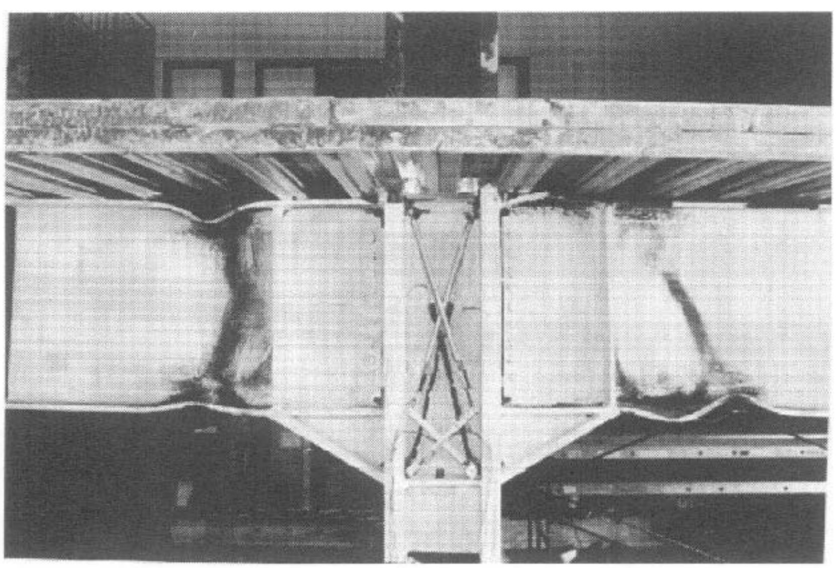


Figure 4.2 NIST/AISC Welded Haunch Test

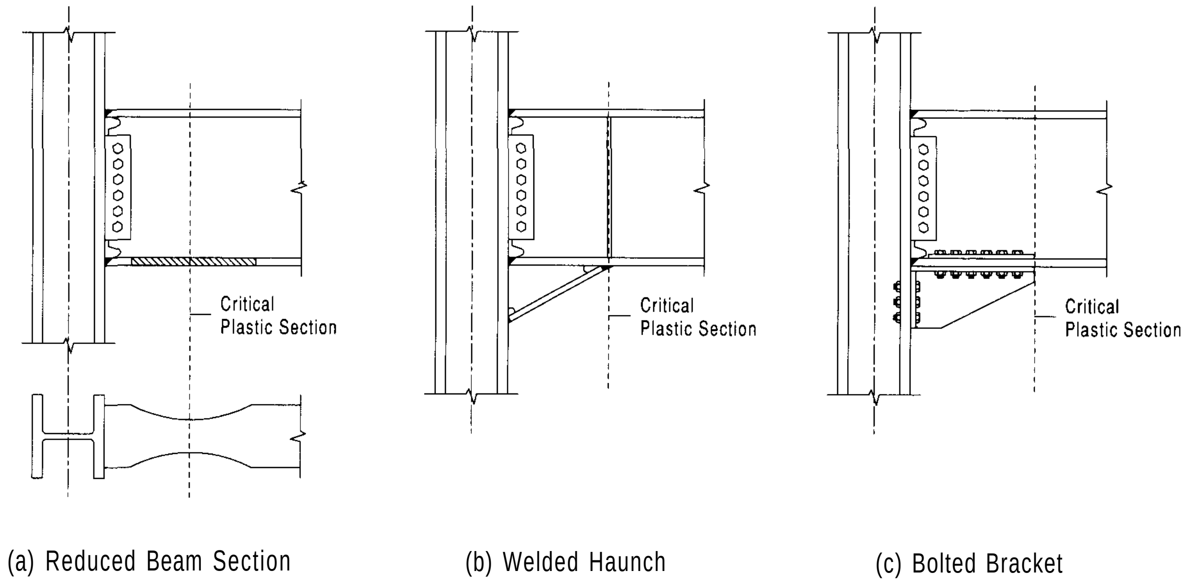


Figure 4.3 Location of Critical Plastic Section

However, as long as the strain hardening factors used for design are calibrated to experimental data using the same critical plastic section, as was done herein, the actual choice for the location of the critical plastic section is rather unimportant. The designer is cautioned that the strain hardening factors used in this Design Guide (see Section 4.3.1) should only be considered valid for the critical plastic section locations listed in Table 4.2.

4.3 Design Forces

Design of a connection modification is based on the limiting moment and the associated shear force at the critical plastic section. The shear force, V_{pd}, and bending moment, M_{pd}, at the critical plastic section are shown in Figure 4.4. Shear force and moment at the column face may be determined by statics knowing the location of the critical plastic section (see Sec. 4.2) and the length of connection modification as shown in Figure 4.4. For example, the moment at the face of the column is given by

$$M_f = M_{pd} + V_{pd}(s_c)$$

4.3.1 Plastic Moment

The plastic moment at a critical section may be determined from the plastic section modulus and the expected material yield strength. The plastic section modulus is based on the assumption that the steel exhibits elastic-perfectly plastic behavior. For very large strains, there is the possibility that the flange material will strain harden and the resulting plastic moment will exceed that computed from the idealized perfectly plastic condition. Thus, the design moment at a plastic critical section, M_{pd}, may be computed

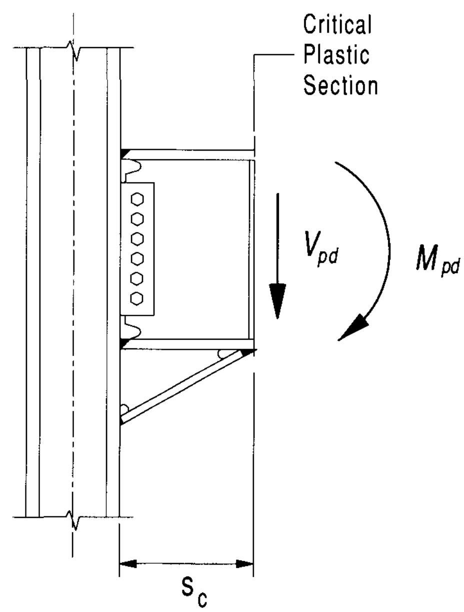


Figure 4.4 Shear Force and Bending Moment at Critical Plastic Section

as

$$M_{pd} = \alpha Z_b F_{ye} \tag{4.2}$$

where

α = factor to account for strain hardening,
Z_b = plastic section modulus of the beam (for the RBS use Z_{RBS} as defined in Section 5), and

F_{ye} = expected yield stress of the beam flanges as determined in Section 4.1.

The strain hardening factor, α, is given for each of the three modifications presented in this Design Guide (see Sections 5, 6, or 7).

4.3.2 Beam Shear

For a beam which is uniformly loaded and rigidly connected at both ends, the shear at the critical plastic section, V_{pd}, is determined from static equilibrium of a free body diagram of the beam section between critical plastic sections, or

$$V_{pd} = \frac{2M_{pd}}{L'} + \frac{wL'}{2} \tag{4.3}$$

where

M_{pd} = design plastic moment given by Eq. 4.2,
L' = beam span between critical plastic sections, and
w = the uniform load on the beam.

If loads other than a uniform load w act on the beam or other end conditions exist, then Eq. 4.3 must be adjusted accordingly. When gravity loads supported by the beam or girder are large, plastic hinges may form within the mid-span region and, in such cases, the location of the plastic section must re-evaluated.

4.3.3 Column-Beam Moment Ratio

The connection modifications described in this Design Guide move the plastic hinge in the beam away from the face of the column. Consequently, the bending moments developed in the beam at the face of the column will be amplified as compared to an unmodified connection, particularly when the modification involves the addition of haunches or other types of reinforcement. These larger beam end moments increase the likelihood of developing flexural plastic hinges in the columns in the region outside of the joint. Current seismic design philosophy for WSMFs generally views the formation of plastic hinges in the columns as less desirable than the formation of plastic hinges in the beams or in the column panel zones. Thus, seismic design codes for WSMFs generally require checking the column-beam moment ratio in order to enforce a "strong column-weak beam" design philosophy. This philosophy reflects the view that formation of column plastic hinges may lead to the development of a soft story, which in turn may lead to story instability.

The degree to which column plastic hinge formation may actually adversely affect the seismic performance of a WSMF is not yet well understood. Research has shown that plastic hinge formation in columns is not always detrimental (Schneider et al. 1993). Further, analyses of WSMFs subject to strong ground motions indicate that simple restrictions on the column-beam moment ratio at a connection, as contained in current seismic codes, may not accurately reflect actual frame behavior (Bondy 1996, Paulay 1997).

Despite uncertainties associated with the strong column-weak beam design philosophy, a simple check on the column-beam moment ratio is advised when modifying an existing WSMF. This check is consistent with current seismic design philosophy for new WSMFs, and can be useful in identifying potential problems with weak columns in existing frames.

The following check on the column-beam moment ratio is recommended:

$$\frac{\Sigma Z_c\left[F_{yc} - (P/A_c)\right]}{\Sigma M_c} \geq 1.0 \tag{4.4}$$

where

Z_c = plastic modulus of the columns above and below the connection,
F_{yc} = specified minimum yield stress for the columns above and below the connection,
P = estimated maximum axial force in columns above and below connection due to combined gravity and lateral loads,
A_c = gross cross-sectional area of the columns above and below the connection, and
M_c = column moments above and below the connection resulting from the development of the design plastic moment, M_{pd}, in each beam at the connection.

With reference to Figure 4.5, ΣM_c can be estimated from the following equations:

$$V_c = \frac{\Sigma\left[M_{pd} + V_{pd}(L - L')/2\right]}{(h_b + d_p + h_t)} \tag{4.5}$$

$$M_{ct} = V_c h_t \tag{4.6}$$

$$M_{cb} = V_c h_b \tag{4.7}$$

$$\Sigma M_c = M_{ct} + M_{cb} \tag{4.8}$$

where

V_c = shear force in columns above and below connection,
h_t = distance from the top of the connection to the point of inflection in the column above the connection,
h_b = distance from the bottom of the connection to the point of inflection in the column below the connection,
d_p = total depth of connection region (depth of beam plus depth of haunches, if present), and
M_{pd}, V_{pd}, L, L' are as previously defined.

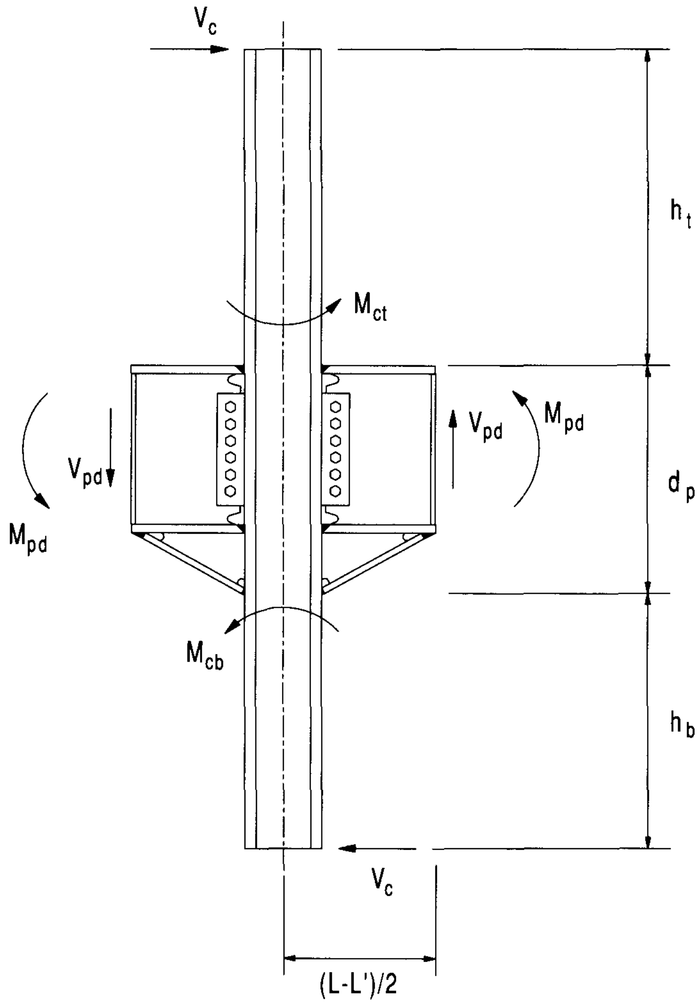


Figure 4.5 Moments for Strong Column Evaluation

The above approach is a simplified version of the approach presented in *Advisory No. 1* (FEMA 1996). While the approach in *Advisory No. 1* accounts for the difference in column shear forces above and below the connection, the simplified approach above assumes the same shear force is present in the columns above and below the connection. Although the approach in *Advisory No. 1* may be somewhat more accurate, the computation of V_c presented in Eq. 4.5 above is simpler to implement, and is considered sufficiently accurate for design purposes considering the numerous other uncertainties involved in the strong column-weak beam design philosophy.

Current seismic design codes for WSMFs contain exceptions to the strong column-weak beam requirement, for which Eq. 4.4 need not be satisfied. These exceptions can be found in the AISC *Seismic Provisions for Structural Steel Buildings* (AISC 1997), and can also be applied in the modification of existing WSMFs. The reader is also referred to the commentary of the *Seismic Provisions for Structural Steel Buildings* for further explanation and background of the strong column-weak beam design requirement.

Strong column-weak beam design requirements for WSMFs first appeared as a code requirement in the U.S. in the 1988 Uniform Building Code (ICBO 1988). Many existing WSMFs designed according to earlier codes may therefore not satisfy Eq. 4.4, even without connection modifications. In such cases, the designer must evaluate the potential impact of column hinging on the seismic performance of the frame. This can be accomplished through inelastic dynamic analysis of the frame using representative ground motion records for the site, including second order effects to evaluate the possibility of story instability. Simpler inelastic pushover analysis may also provide insight into the potential impact of column hinging. If analysis indicates that column hinging may lead to frame instability, the designer should consider alternative frame modifications such as the addition of bracing or the addition of energy dissipation devices. Further, for frames in which column hinging is of concern, the RBS modification may be preferable to the use of haunches or other types of reinforcement. The RBS modification reduces beam end moments as compared to an unmodified or reinforced connection, and can be used to advantage to reduce the possibility of column hinge formation.

4.4 Connection Modification Objectives

The objective of the connection modifications described in this Design Guide is to improve the performance of an existing WSMF in future earthquakes. The 1994 Northridge earthquake demonstrated that connections in existing WSMFs may be vulnerable to premature fracture. In this earthquake, no WSMF buildings collapsed and no lives were lost as a result of these connection fractures. However, these fractures lead to significant economic losses associated with the inspection and repair of damaged connections and the consequent disruption to building occupants and activities.

The safety implications of connection damage in WSMFs are still not clear. The absence of collapses in the Northridge earthquake provides at least some reassurance that a WSMF may be capable of sustaining significant connection damage without endangering life safety. There may be several reasons for this, including residual strength in damaged connections, partial moment restraint provided by nominally "pinned" beam-to-column connections, beneficial effects of floor slabs, beneficial effects of column continuity, reduction in seismic demands due to building period shifts resulting from connection damage, and other factors. Nevertheless, the significance of connection damage in earthquakes which have magnitude, duration, or frequency content that differ from the Northridge earthquake may be greater.

While the safety implications of connection damage in WSMFs are not yet clear and may be debatable, it appears clear that such damage can be quite costly. The overall objectives then of modifying connections in existing WSMFs are to mitigate both the economic impact and potential life safety concerns associated with connection damage in future earthquakes.

The ability of a beam-to-column connection to withstand earthquake demands without failure has commonly been measured by the connection's plastic rotation capacity. Actual plastic rotation demands in WSMFs subject to earthquake motions are difficult to assess, and one must resort to inelastic time-history analysis or shaking table tests to provide estimates. As part of the SAC Phase 1 research, inelastic time-history analyses were conducted on 10 WSMF buildings that experienced varying degrees of connection damage in the Northridge earthquake (SAC 1995). Analyses of these buildings, which ranged from 2 to 17 stories in height, indicated that the plastic rotation demands resulting from the Northridge Earthquake ground motions were in the range of 0.01 radian to 0.015 radian at the most severely loaded connections. The connection damage experienced in these buildings suggests that the pre-Northridge connection detail is often incapable of sustaining these levels of plastic rotation without failure. Experiments conducted on pre-Northridge connections (SAC 1996) confirmed that fracture generally occurred at plastic rotation levels less, and often significantly less, than about 0.01 radian to 0.015 radian. This same SAC analytical study also examined the response of the ten buildings to a variety of other, potentially more damaging ground motions. It was found that maximum plastic hinge rotations on the order of 0.015 radian to 0.025 radian were obtained when the buildings were subjected to a suite of actual ground motion records roughly consistent with a response spectra with a 10 percent probability of exceedance in 50 years. While ongoing research suggests that this range may not be conservative for all conditions, it appears to be reasonable over a wide range of practical design situations.

Based on currently available evidence, *Interim Guidelines* (FEMA 1995) and *Advisory No. 1* (FEMA 1996) recommend that connections in new steel moment frames be capable of providing at least 0.03 radian of plastic rotation without failure. Further, these documents provide suggested connection details believed capable of providing this level of plastic rotation. As compared to the pre-Northridge connection, these improved connections generally implement improved welding practices combined with connection design enhancements.

Many of the connection details suggested in the *Interim Guidelines* and *Advisory No. 1* for new construction can potentially be applied to the modification of existing WSMF connections. This approach should lead to connection performance similar to that anticipated for new construction, i.e., connections capable of developing at least 0.03 radian of plastic rotation. However, many of the connection details intended for new construction may be prohibitively expensive when applied to existing buildings due to problems of limited access (e.g., concrete slab), fire and fume hazards associated with welding in an existing building, etc. Nevertheless, employing new construction type connection details for modifying existing WSMF connections is an option open to the designer.

The objective of the connection modifications for existing WSMFs presented in this Design Guide is to provide a significant improvement in connection performance as economically as possible. Experiments on the recommended connection modifications, i.e., the welded haunch, the bolted bracket, and the RBS modifications, indicate that the modified connections should generally be capable of developing at least 0.02 radian of plastic rotation. While not meeting new construction standards, these modified connections will provide a significant improvement in performance compared to existing pre-Northridge connections. The use of these modified connections should reduce potential economic losses and mitigate safety concerns for existing WSMFs in future earthquakes. In the judgment of the writers, modified connections capable of developing at least 0.02 radian of plastic rotation provide a reasonable basis for the seismic rehabilitation of many buildings constructed with WSMFs. However, under some conditions a higher level of plastic rotation capacity may be needed and may be appropriate in the rehabilitation of a WSMF. Examples of such conditions may include buildings designed for large pulse-like near field demands, buildings on soft soils, irregular buildings, essential facilities, and others. When such conditions are present, special studies may be needed to better define WSMF connection requirements. As described earlier, if higher plastic rotation capacities are desired, the new construction details described in the *Interim Guidelines* (FEMA 1995) and *Advisory No. 1* (FEMA 1996) provide an alternative approach. It should be recognized that regardless of the detail chosen for connection modification, some damage should still be expected in a very strong earthquake. Local buckling of beam flanges generally develops at large plastic rotations. Should these high levels of plastic rotation be experienced in a very strong earthquake, costs would likely be incurred to repair the beam local buckles and other potential damage. Thus, modifying connections in an existing WSMF does not preclude damage in future earthquakes. However, modified connections should be capable of sustaining larger earthquakes with less damage.

When evaluating performance objectives for the rehabilitation of an existing WSMF, the designer is also encouraged to consult FEMA 273, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (FEMA 1998).

4.5 Selection of Modification Method

Of the three connection modification methods described in this Design Guide, choosing the most suitable method for a particular project will depend on a number of project specific factors. Consequently, no general recommendation can be provided herein on a preferred method. Nonetheless, the designer should consider the potential advantages and disadvantages of each method prior to making a choice. Some of the issues that may affect the choice of a connection modification method include plastic rotation requirements, reliability of the modified connection, cost, constructability issues, the ability to satisfy strong column-weak beam requirements, and other factors.

Each of the three connection modification methods have developed plastic rotation capacities of at least 0.02 radian in cyclic loading tests (Section 3.2). However, some modification methods provided higher levels of plastic rotation than others. For example, the welded haunch modification in the presence of a composite slab and the bolted bracket modification each developed in excess of 0.03 radian of plastic rotation capacity. On the other hand, the bottom flange RBS only developed on the order of 0.02 to 0.025 radian of plastic rotation. Thus, the welded haunch and bolted bracket may offer a higher level of performance and reliability.

The welded haunch offers the advantage that no modifications are required at the existing top flange weld, minimizing or eliminating the need for removing a portion of the concrete slab. The bolted bracket requires the installation of top flange reinforcement, necessitating the removal and replacement of a portion of the slab. The bolted bracket, on the other hand, offers the advantage of eliminating field welding. Both the welded haunch and bolted bracket will increase the bending moment transferred from the beam to the column as compared to an unmodified connection. The RBS modification, on the other hand, reduces the moment transferred to the column, and may therefore be advantageous in situations where strong column-weak beam requirements are critical. Further, the space required by the welded haunch and bolted bracket may cause interference problems in situations where little space is available below the beam. The RBS modification requires no additional space above or below the beam. Finally, cost is an important factor affecting the choice of a modification method. Cost issues are discussed in Chapter 8.

Chapter 5
DESIGN OF REDUCED BEAM SECTION MODIFICATION

Based on a review of experimental data on RBS connections, both for new construction and for modification of existing connections (see Section 3), it is clear there is no single approach for designing and detailing these connections. For the RBS cutout, there are a variety of shapes and sizes which can be used, as well as the possibility of cutting the RBS in both the top and bottom flanges or in the bottom flange only. Beyond the size, shape and location of the RBS cutouts, there is a further variety of design and detailing options which may enhance connection performance. This section addresses these various design and detailing options and recommends a procedure for designing the radius cut RBS modification.

5.1 Recommended Design Provisions

When considering RBS modifications of an existing WSMF connection, a number of options are available to the designer, including:

- Use of RBS cutout in bottom flange only, or in both top and bottom flanges;
- Shape of RBS cutout (constant cut, tapered cut, radius cut, or other);
- Dimensions of RBS cut (distance from face of column to start of cut, length of cut, depth of cut, etc.);
- Replacement of existing weld metal with higher toughness weld metal;
- Removal or seal welding of steel backing;
- Removal of weld tabs;
- Addition of continuity plates, if not already present;
- Replacement of the existing bolted web connection with a welded web connection;
- Addition of supplemental beam lateral brace at the RBS cut;
- Addition of supplemental reinforcement at the beam-to-column connection (ribs, cover plates, etc.).

The designer must make a decision on each of the above issues. The choices made on these will impact both the cost and the performance of a modified connection. Unfortunately, there is insufficient data to support a firm recommendation on each item above. Rather, the data provides guidance on the minimum modifications needed to achieve at least a reasonable degree of performance improvement, and what additional modifications are likely to lead to further enhancement of the ductility and reliability of the modified connection. Consequently, in this section, minimum recommended modifications are presented first. This is followed by suggestions for additional modifications to further enhance connection performance.

5.1.1 *Minimum Recommended RBS Modifications*

This section contains recommendations for the minimum modifications to an existing WSMF connection that are likely to provide a significant improvement in the connection's plastic rotation capacity. These recommendations are based on the tests of RBS modified connections summarized in Table 3.4. Based on these tests, the following minimum modifications are recommended:

1. Provide an RBS cut in the beam bottom flange, and
2. Replace the existing top and bottom beam flange CJP groove welds with high toughness weld metal, and
3. At the bottom flange groove weld, remove the backing and weld tabs; repair any weld defects and provide a reinforcing fillet, and
4. At the top flange groove weld, remove weld tabs and weld backing to face of column.

As described earlier, the test data suggests that the bottom flange RBS, without significant weld modifications, is inadequate to prevent early brittle fracture of existing low toughness welds. Thus, it is recommended that, at a minimum, the bottom flange RBS is combined with the replacement of both the top and bottom flange welds. Tests suggests that this level of modification permits the development of plastic rotations on the order of 0.02 radian to 0.025 radian. The following sections provide more specific recommendations.

5.1.2 *Size and Shape of RBS Cut*

Typical shapes of RBS cuts used in past research are illustrated in Figure 2.2, and include the constant cut, the tapered cut, and the radius cut. The constant cut may offer the advantage of simplified fabrication. The tapered cut, on the other hand, is intended to match beam strength to the shape of the moment diagram. Both of these types of RBS cuts have shown good performance in laboratory tests, although a few have experienced fractures within the reduced section after developing large plastic rotations. These fractures have occurred at changes in section within the RBS, for example at the minimum section of the tapered RBS. These changes of cross-section presumably introduce stress concentrations that can lead to fracture within the highly stressed reduced section of the beam. The radius cut RBS appears to minimize stress concentrations and no fractures within the RBS have been reported in radius cut RBS tests. Further, the radius cut is still relatively simple to fabricate. Consequently, for modification of existing WSMF connections, the radius cut RBS is recommended.

Figure 5.1 illustrates key dimensions that characterize the radius cut RBS. These include *a*, the distance from the face of the column to the start of the cut; *b*, the total length of the cut; and *c*, the depth of the cut. The radius of the cut, *R*, is determined by *b* and *c*, based on the geometry of a circular arc, as shown in Figure 5.1. The depth of cut is also often characterized by the percentage flange reduction, which is computed as $2c/b_f$ X100, where b_f is the original flange width. The center of the RBS is treated as the critical plastic section for connection design purposes. The distance from the face of the column to the critical plastic section is designated as s_c and is computed as $a + (b/2)$.

In past tests, the dimensions *a* and *b* have generally been chosen based on the judgment of the researchers. In general, it appears preferable to keep these dimensions as small as possible in order to minimize the growth of moment from the hinge located in the RBS back to the face of the column. The dimension *a* should be large enough, however, to permit stress in the reduced section of the beam to spread uniformly across the flange width at the face of the column. Similarly, the dimension *b* should be large enough to avoid excessive inelastic strains within the RBS. Thus, the dimensions *a* and *b* should be chosen considering these opposing requirements. Based on an evaluation of past test data, the following suggestions are made for choosing these dimensions:

$$a \approx (0.5 \text{ to } 0.75)\, b_f \tag{5.1}$$

$$b \approx (0.65 \text{ to } 0.85)\, d \tag{5.2}$$

where b_f and *d* are the beam flange width and beam depth, respectively. Examination of RBS test data indicates that successful connection performance has been obtained for a wide range of values of *a* and *b*. Consequently, a great deal of precision in choosing these values does not appear to be justified.

The remaining dimension that must be chosen when sizing the RBS is *c*, the depth of the cut (Figure 5.1). The value of *c* will control the maximum moment developed within the RBS, and therefore will control the maximum moment at the face of the column. In establishing a method for choosing the value of *c*, the following assumptions have been made:

- The maximum moment developed at mid-length of the RBS, M_{pd}, is equal to 1.1 times the plastic moment of the reduced section. Thus,

$$M_{pd} = 1.1\, Z_{RBS} F_{ye} \tag{5.3}$$

where

M_{pd} = design moment at the critical plastic section (mid-length of RBS)
Z_{RBS} = plastic section modulus at minimum section of the RBS
F_{ye} = expected yield stress of beam flange

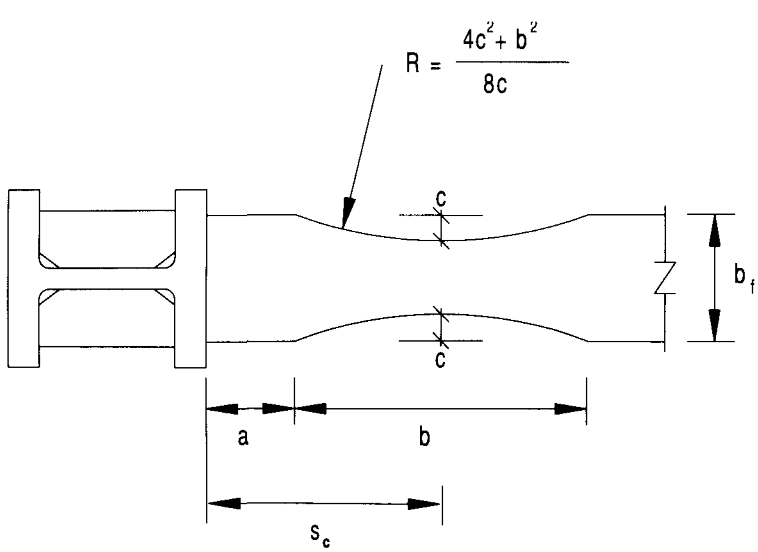
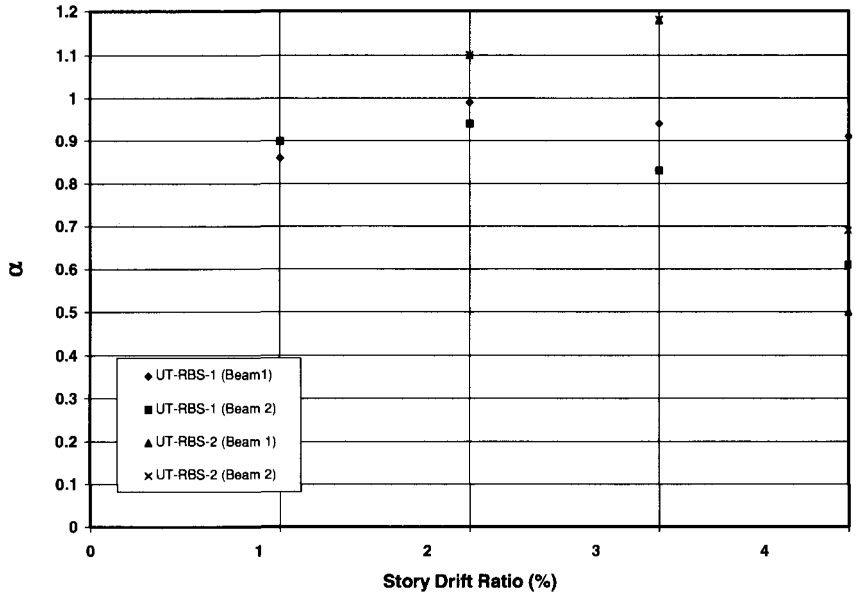
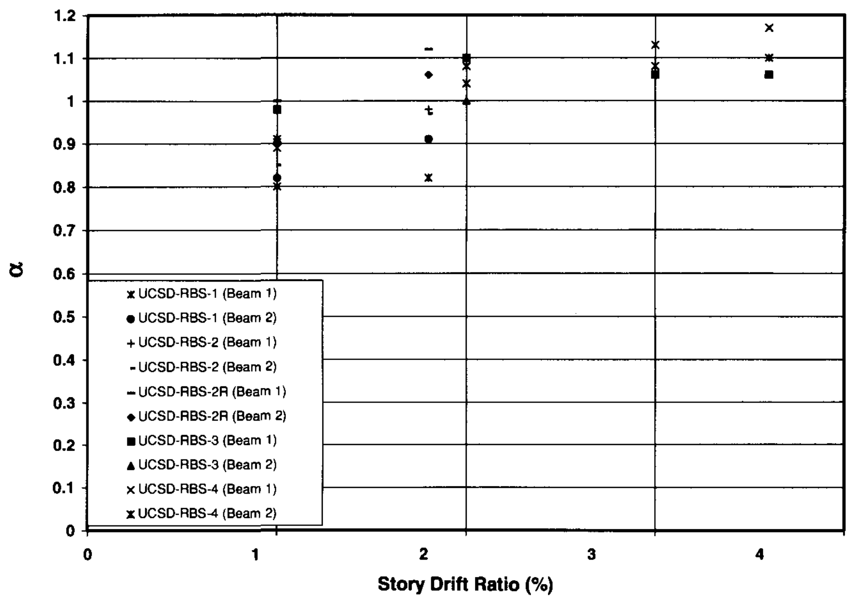


Figure 5.1 Geometry ofRadius Cut RBS



(a) W30 × 99 Beams



(b) W36×150 Beams

Figure 5.2 Relationship Between Story Drift Ratio and a for Bottom Flange RBS Specimens

The 1.1 factor in Eq. 5.3 is the α factor described in Section 4.3.1 and accounts for strain hardening and other factors that increase the moment at the center of the RBS beyond the plastic moment, $Z_{RBS}F_{ye}$. The value of 1.1 was established from experimental data on connections in the NIST/AISC test program that were modified using a bottom flange RBS. This data is shown in Figure 5.2 for test specimens with W30×99 beams and with W36×150 beams. The value of a plotted in this figure was computed by taking the maximum moment measured at the center of the RBS at various story drift ratios and dividing by the plastic moment at the center of the RBS, $Z_{RBS}F_{ye}$, where F_{ye} was taken as the actual yield stress of the test beam based on tensile coupon tests. The maximum moment at the center of the RBS was taken as the maximum of either the positive or negative moment measured at various story drift ratios. Consequently, for specimens with composite floor slabs, the α values plotted in Figure 5.2 implicitly include the effects of the floor slab on the moment developed within the RBS. While Figure 5.2 shows considerable variability in the measured α values, a value of 1.1 was chosen for design purposes as a reasonable upper bound for the majority of test specimens.

- Once M_{pd} has been computed, the moment at the face of the column, M_f, can be computed by considering static equilibrium of the beam span between critical plastic sections, and also of the beam segment between the critical plastic section and the face of the column as indicated by Eq. 4.3. For a uniform gravity load w, as shown in Figure 5.3, the moment at the face of the column can be approximated as follows:

$$M_f = \left[1 + \frac{2s_c}{L'}\right]M_{pd} + \frac{wL's_c}{2} \tag{5.4}$$

where

M_f = maximum moment at face of column
s_c = distance from face of column to center of RBS
L' = beam span between centers of RBS cuts
w = uniformly distributed gravity load on beam.

Note that Eq. 5.4 neglects the influence of the portion of the gravity load within the length s_c at each end of the beam. This simplifies the calculation and introduces little error.

- The maximum practical cutout is approximately 50 percent of the flange width (corresponding to c = 0.25 b_f). This is based on the judgment of the writers and on flange reduction values used in past testing. The largest flange reduction used in past RBS tests for new construction applications (Table 3.1) appears to be 55 percent. For tests on RBS modifications of existing connections, a maximum flange reduction of 50 percent has been used. Flange reduction significantly larger than 50 percent may risk impairing the stability of the beam, and is not recommended without experimental verification.

As noted earlier, the amplification of the plastic moment in the RBS can be minimized by keeping the dimensions a and b as small as possible, within the bounds suggested in Eqs. 5.1 and 5.2. If this moment amplification becomes too large, much of the benefit of the RBS is negated.

Equation 5.3 requires the computation of Z_{RBS}, the plastic section modulus at the minimum section of the RBS. Figure 5.4 illustrates a cross-sectional view of the RBS, showing the cut regions of depth c at the bottom flange only. This figure also shows the location of the plastic neutral axis for this section. For the cross-section shown in Figure 5.4, i.e., for an RBS with bottom flange cutouts only, the plastic section modulus can be computed as follows:

$$Z_{RBS} = Z_b - \frac{(ct_f)^2}{t_w} - ct_f(d - t_f) \tag{5.5}$$

where

Z_b = plastic section modulus for full beam cross-section (i.e., without flange cutouts)
all other variables are as shown in Figure 5.4

Equation 5.5 assumes the plastic neutral axis remains within the web. This will be the normal case, and can be checked as indicated in Figure 5.4.

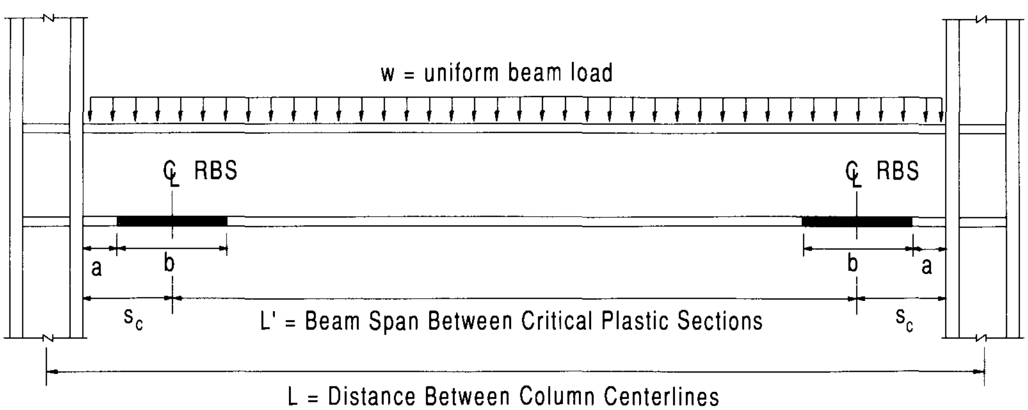


Figure 5.3 Typical Beam Span with Bottom Flange RBS Cutout

Using the equations given above, it is possible to compute the depth of cut c needed to limit the moment at the face of the column, M_f, to some desired value considered to be safe for the conditions involved. In making this calculation, it is convenient to normalize all quantities with respect to the expected beam flange yield stress, F_{ye}. Thus, the moment at the face of the column can be written as follows:

$$M_f = \eta Z_b F_{ye} \tag{5.6}$$

In this equation, η represents the maximum moment at the face of the column divided by the plastic moment of the beam.

Substituting Eqs. 5.3 and 5.6 into Eq. 5.4, leads to the following:

$$\eta Z_b F_{ye} = \left(1 + \frac{2s_c}{L'}\right)1.1 Z_{RBS} F_{ye} + \frac{wL's_c}{2} \tag{5.7}$$

Finally, by dividing both sides by $F_{ye}Z_b$, the following equation results:

$$\eta = \left(1 + \frac{2s_c}{L'}\right)1.1\frac{Z_{RBS}}{Z_b} + \frac{wL's_c}{2Z_bF_{ye}} \tag{5.8}$$

Ideally, the RBS dimensions should be chosen to keep the value of η as small as possible, i.e., to keep the moment at the face of the column as low as possible. For new construction applications, where cutouts are provided in both the top and bottom flanges, it is possible to achieve values of η in the range of 0.85 to 0.95. Thus, the maximum moment at the face of the column will be limited to 85 to 95 percent of the beam's plastic moment.

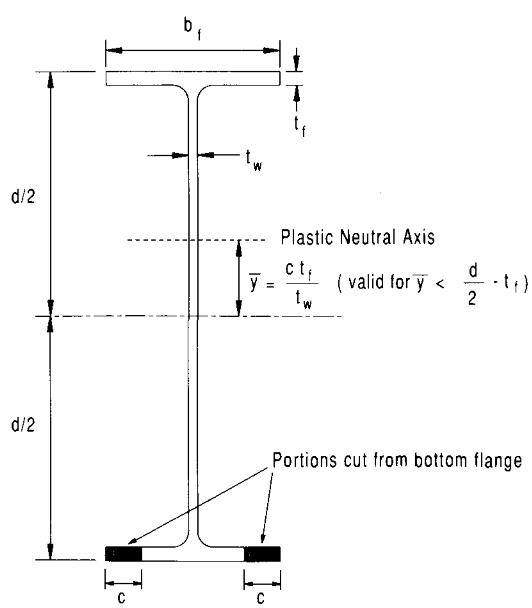


Figure 5.4 Cross-Section of RBS with Bottom Flange Cutouts

For modification of existing connections, where cutouts are provided in the bottom flange only, larger values of η must be tolerated. For the tests on RBS modifications listed in Table 3.4, 50 percent flange reductions were provided in the bottom flange. Use of Eq. 5.8 leads to values of η equal to 1.01 for the W30×99 test beams and 1.04 for the W36×150 test beams listed in Table 3.4. These specimens provided plastic rotations of 0.02 radian to 0.025 radian when the bottom flange RBS was combined with the use of high toughness weld metal in both top and bottom flanges.

Based on the limited test data for RBS modifications, it is recommended that the depth of the bottom flange RBS cutout be chosen to provide a value of η not to exceed 1.05. If this value of η cannot be achieved with a 50 percent flange reduction, then a bottom flange RBS modification is not recommended. A value of η equal to 1.05 implies the maximum moment at the face of the column will be on the order of 1.05 times M_p of the beam. The limited test data on RBS modifications is inadequate to determine if this level of moment can be tolerated by pre-Northridge type connections with high toughness welding on a consistent and reliable basis. A value of η equal to 1.05 should therefore be considered an upper bound. Smaller values of η are preferred. Consequently, the largest possible flange reduction should be used for the RBS in order to minimize the moment at the face of the column.

Based on the discussion above, it is recommended that the bottom flange RBS be sized for a 50 percent flange reduction. This will result in the greatest reduction in the moment at the face of the column. Consequently, the design process for the bottom flange RBS can be summarized as follows:

STEP 1 Choose $c = 0.25\ b_f$ (i.e., 50 percent flange reduction).
STEP 2 Compute Z_{RBS} from Eq. 5.5.
STEP 3 Compute η from Eq. 5.8.
STEP 4 If η is less than 1.05, then the RBS dimensions are satisfactory. If η exceeds 1.05, then use RBS cutouts in both the top and bottom flanges, or use some other type of connection modification, e.g., haunches.

In Eqs. 5.3, 5.6 and 5.7 above, F_{ye} represents the expected yield stress of the beam flanges. F_{ye} cancels out of the calculations in going from Eq. 5.7 to Eq. 5.8 except in the gravity load term. This procedure implies that both the demand on the connection imposed by plastic hinge formation in the RBS (Eq. 5.3), as well the strength of the connection (Eq. 5.6), are computed with respect to the expected yield stress of the beam. Thus, the limiting moment expected at the face of the column is $\eta M_p = \eta Z_b F_{ye}$

Consequently, this design procedure provides no absolute limit on the maximum moment imposed by the beam on the face of the column. Rather, the procedure limits the maximum moment at the face of the column relative to the actual plastic moment of the beam. For example, suppose the beam is specified to be of A36 steel, and the dimensions of the RBS are chosen based on $\eta = 1.05$. If the actual yield stress of the beam is 40 ksi, then the maximum moment expected at the face of the column is approximately $1.05 \times Z_b \times 40$ ksi. On the other hand, if the actual yield stress of this A36 beam is 55 ksi, then the maximum moment expected at the face of the column is approximately $1.05 \times Z_b \times 55$ ksi.

Since the design procedure for sizing the RBS provides no absolute limit on the maximum moment at the face of the column, this procedure should only be applied in cases where columns have a minimum specified yield stress of 50 ksi or higher, and where the beams were specified as A36 steel. Even then, the performance of the modified connection may be poorer in cases where the actual yield stress of the beam is substantially higher than 36 ksi.

As an alternative to the design procedure described above, design guidelines presented in *Advisory No. 1* (FEMA 267A 1996) recommend that an RBS be sized to provide an absolute limit to the stress at the face of the column, equal to ninety percent of the minimum specified yield stress of the column. Based on research currently underway, this stress limit will likely be increased in the future. Nevertheless, the stress limit recommended by *Advisory No. 1* cannot, in general, be achieved with a bottom flange only RBS. Thus, the designer is cautioned that the procedure described in this section for sizing a bottom flange RBS will not, in general, conform to the guidelines presented in FEMA-267A. In order to meet the stress limits of *Advisory No. 1*, the designer should consider an alternative connection modification, such as the use of a bottom flange haunch or other reinforcement techniques, or a combination of an RBS and connection reinforcement. The design of the test specimens incorporating bottom flange RBS modifications (Table 3.4), upon which these guidelines are based, did not conform to the recommendations of *Advisory No. 1* but still developed plastic rotations on the order of 0.02 radian to 0.025 radian.

Regardless of the basis for choosing the dimensions of the RBS cut, it is important that a smooth cut be provided. The RBS cut is normally made by thermal cutting. The cut should be as smooth as possible, avoiding nicks, gouges, and other discontinuities. After the cut is made, the surface should be ground smooth, with the grinding done in a direction parallel to the beam flange. This avoids grind marks perpendicular to the beam flange, i.e., perpendicular to the direction of stress, which can act as stress risers. Even if the thermal cut surface appears to be very smooth, grinding is still recommended as it will remove any surface material adversely affected by the heat of the cutting operation.

5.1.3 Flange Weld Modifications

Tests of pre-Northridge connections modified with a bottom flange only RBS showed very poor performance when existing E70T-4 weld metal was left in place at the beam flange groove welds (see Table 3.4). Significantly improved performance was achieved when the existing E70T-4 weld metal was replaced with a higher toughness weld metal. Consequently, as part of the minimum RBS modification, it is recommended that existing weld metal at the top and bottom flange groove welds be removed and replaced with a weld metal exhibiting improved notch toughness. The minimum toughness needed for groove welds in this application has not yet been quantified. A number of successful tests have employed weld metal with a minimum specified Charpy V-Notch (CVN) value of 20 ft-lb at -20°F. Thus, pending further research and based on available test data on RBS connections, it is recommended that the replacement weld metal provide a minimum specified tensile strength of 70 ksi, and a minimum specified CVN value of 20 ft-lb at -20°F. Past tests on RBS connections, both for new construction and for modification of existing connections, have generally employed the self shielded flux cored arc welding process (FCAW), using either the E70TG-K2 or E71T-8 electrodes. Both of these electrodes provide a minimum specified CVN of 20 ft-lb at -20°F. A number of other FCAW electrodes are also available which provide this minimum CVN value. In addition, successful tests on other types of connections have employed the shielded metal arc welding (SMAW) process using an E7018 electrode (Kaufmann et al. 1996). The final choice of welding process and electrode should be made in consultation with the fabricator that will perform the work based on the conditions involved in the actual building.

Removal of the existing weld metal is normally accomplished by air carbon arc cutting (CAC-A), commonly called arc gouging, or by grinding. It is important to remove all of the existing weld metal. The weld removal process, however, should be executed with care in order to avoid removing excessive base metal from the column or beam, and to avoid damaging the column and beam. Any discontinuities in the face of the column flange or in the beam flange should be repaired. If the existing weld metal was removed by gouging, grinding of the gouged surface may be needed to provide a surface suitable for welding per AWS D1.1-98 (AWS 1998).

Prior to rewelding, the groove weld joint dimensions should conform to the requirements shown in Figure 3.4 of AWS D1.1-98, or should be qualified by test, as also permitted by AWS D1.1-98.

Prior to rewelding, it is also recommended that the weld access holes be modified, if needed. Access hole geometry should conform to the requirements shown in Figure 5.2 of AWS Dl.1-98. The surfaces of the access hole should be ground smooth per Section M2.2 of the 1993 AISC-LRFD Specification. Tests on moment connections have frequently shown fractures initiating at the point where the access hole meets the inside face of the flange. Consequently, a smooth transition between the access hole and the inside face of the beam flange is particularly important. The proper size, geometry and finish of the access holes will contribute to enhanced connection performance by permitting proper access for the welder, by minimi/ing stress concentrations caused by the access hole, by alleviating triaxial states of stress (Blodgett 1998), etc.

After the new groove welds are completed, it is recommended that the weld tabs be removed at both the top and bottom flanges, and the edges of the groove welds ground smooth. This will minimize any potential notches introduced by the presence of the weld tabs, or by discontinuities contained in the weld metal within the run-off regions. In addition, it is recommended that the bottom flange steel backing be removed and a reinforcing fillet be placed at the base of the groove weld. This requirement is intended to eliminate the notch effect produced by left-in-place steel backing, and to permit better inspection and ultrasonic testing of the weld. Care should be taken to not damage the base metal when removing the backing. Any pits, gouges, discontinuities and slag pockets discovered upon removal of the backing should be ground out prior to rewelding. Finally, at the top flange groove weld, the steel backing should be seal welded to the face of the column using a fillet weld. Analysis has indicated that the notch effect of the steel backing is not as severe at the top flange, and that welding the steel backing to the column further reduces the notch effect (Yang and Popov 1995). It is also acceptable to remove the top flange steel backing. However, leaving the top steel backing in-place and welding it to the column is likely to be less costly than removing it.

It is recommended that 100 percent of groove welds in modified connections be ultrasonically tested. Minimum acceptance criteria are recommended to be in conformance with Table 5.2 of AWS D1.1-98.

If reliable records are available that indicate the original welds in the existing moment connections were made with an electrode providing a minimum specified CVN of 20 ft.-lb. at -20°F, then removal of the existing weld metal is not necessary. All other measures recommended above, however, should still be followed. That is, weld tabs and the bottom flange steel backing should be removed, the top flange steel backing should be welded to the column, and 100 percent of all groove welds in modified connections should be ultrasonically tested.

All welding operations involved in a connection modification should conform to the requirements of AWS D1.1-98, including preheat and the use of written welding procedure specifications. Welding at an existing connection, in general, may be substantially more difficult and costly than welding during new construction. Some of the factors to be considered in modification welding include more difficult access for the welder and inspector, poor welding position, higher levels of restraint, increased fire danger, control of fumes from welding and cutting, the presence of paint, etc. Consequently, all welding related operations involved in a connection modification should be carefully developed by individuals experienced in such operations. Consultation between the designer, fabricator and welding specialists is recommended to develop methods and procedures for welding modifications of existing connections. Additional useful information on welding moment connections can be found in a number of references, including (FEMA 1995; FEMA 1997; and Blodgett, Funderburk and Miller 1997).

Removal and replacement of the top flange weld, as recommended above, is likely to necessitate removal of at least a small potion of the floor slab. This will generally be needed to permit proper access to the weld joint by the welder and inspector, and to permit proper preheating. When replacing the removed portion of the floor slab, the designer should consider leaving a small gap between the slab and the face of the column. This will help minimize composite action at the joint, thereby reducing demands on the bottom flange weld. Successful tests on one-sided RBS connections by Tremblay et al. (1997) employed composite slabs with a 1 inch gap at the face of the column. Such techniques may enhance the performance of the modified connection.

5.1.4 Techniques to Further Enhance Connection Performance

In the previous section, minimum requirements for an RBS modification of existing pre-Northridge moment connections were presented. A very limited number of tests employing these minimum modifications suggest that plastic rotations on the order of 0.02 radian to 0.025 radian can be achieved with these minimum modifications. However, the test result database on RBS modified connections is very small and does not cover all of the possible variables that may affect connection performance. The test database at this time is also insufficient to assess the reliability of RBS modified connections.

Further enhancement of the plastic rotation capacity and reliability of an existing connection may be possible by employing additional modifications to the connection. As indicated by Table 3.1, there is a substantially larger testing database on RBS connections for new construction applications. This database shows that the majority of RBS connections for new construction applications have shown excellent performance, developing plastic rotations on the order of 0.03 radian or better on a consistent basis. This database can therefore be used as a basis for selecting additional potential modifications for an existing connection, provided the existing connection conditions can be made comparable to new construction conditions.

The above discussion suggests that to further enhance the performance of an existing pre-Northridge connection, additional modifications can be employed which approach the details used for new construction applications of RBS connections. Thus, the remainder of this section discusses additional modifications that are likely to further enhance connection performance. Note that these modifications are in addition to those already recommended in Section 5.1.1. Thus, in all cases, a bottom flange RBS should be provided along with the welding modifications recommended in Section 5.1.1.

For each of the additional modifications listed below, there is insufficient data to quantify the benefits of each modification. The only thing that can be currently inferred from the available data is that each of these modifications should improve the plastic rotation capacity and reliability of the connection. The cost and potential benefits of each of these modifications must be considered on a case specific basis.

Use of RBS Cuts in Both Top and Bottom Flanges. All of the tests on RBS connections for new construction applications have employed RBS cuts in both the top and bottom flanges. The use of RBS cuts in both flanges permits a substantially greater reduction in bending moment at the face of the column. For typical beam sizes and bay widths, a 50 percent RBS cut in the bottom flange only will limit the maximum moment at the face of the column to a value on the order of 100 to 105 percent of the beam's plastic moment (i.e., η = 1.0 to 1.05 in Eq. 5.8). On the other hand, providing a 50 percent RBS cut in both flanges can limit the maximum moment at the face of the column to a value on the order of 85 to 95 percent the beam's plastic moment (i.e., η = 0.85 to 0.95 in Eq. 5.8). This reduced moment at the face of the column is likely to be highly beneficial to the connection performance.

The minimum recommended modifications presented in Section 5.1.1 require an RBS cut in the bottom flange only. Discussions with fabricators and erectors have indicated that cutting an RBS into the top flange would likely necessitate removal of the floor slab in the region of the cut. In order to avoid the cost of removing a large portion of the floor slab, an RBS cut was not required in the top flange. Nevertheless, connection performance may be enhanced considerably by providing the RBS cut in both the top and bottom flanges. Consequently, a designer should consider this possibility. Consultation with a fabricator may be beneficial on a case specific basis to evaluate the actual costs involved in providing an RBS cut in both flanges. Consideration should also be given to making RBS cuts to both the top and bottom flanges for the most critical moment connections, while using bottom only RBS cuts for the balance of the moment connections.

If an RBS cut is to be provided in both the top and bottom flanges, the guidelines for choosing the size and shape of the RBS cuts provided in Section 5.1.1 are still applicable. That is, a circular cut RBS is recommended as shown in Figure 5.3, with the dimensions a and b chosen in accordance with Eqs. 5.1 and 5.2. The depth of the cut can still be chosen based on Eq. 5.8. However, Equation 5.5 can no longer be used to compute the plastic section modulus at the minimum section of the RBS, i.e., Z_{RBS}. Eq. 5.5 is only applicable when the RBS cut is provided in the bottom flange only.

When RBS cuts are provided in both the top and bottom flanges, with a cross-section as shown in Figure 5.5, the plastic section modulus at the reduced section can be computed as follows:

$$Z_{RBS} = Z_b - 2ct_f(d - t_f) \tag{5.6}$$

where

Z_{RBS} = plastic section modulus at minimum section of RBS

Z_b = plastic section modulus for full beam cross-section (i.e., without flange cutouts)

and all other variables are as shown in Figure 5.5.

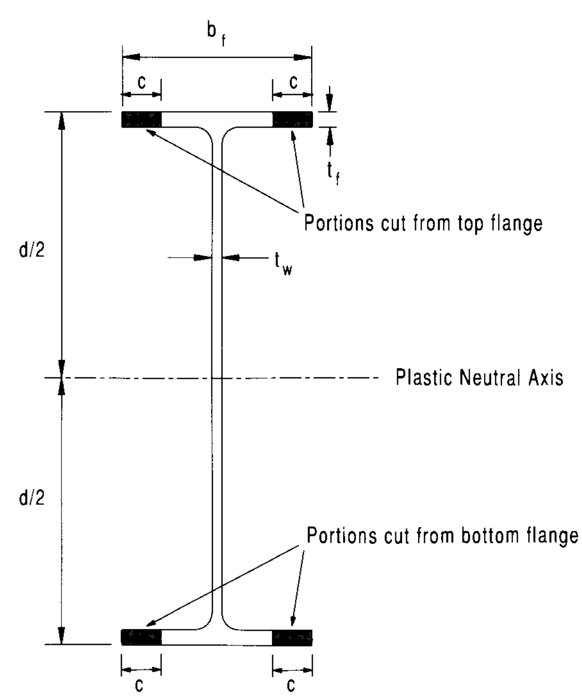


Figure 5.5 Cross-Section of RBS with Top and Bottom Flange Cutouts

Using a value of Z_{RBS} computed from Eq. 5.6, the value of 17 can be computed from Eq. 5.8. For RBS cuts in both the top and bottom flanges, values of η in the range of 0.85 to 0.95 can often be achieved. These values are similar to those achieved in RBS test specimens for new construction applications.

As was the case when the RBS cut was provided in the bottom flange only, even when the cut is provided in both flanges, the designer should consider providing the maximum practical cut, corresponding approximately to a 50 percent flange reduction (i.e., $c = 0.25b_f$). This will provide for the maximum reduction in moment at the face of the column. Note, however, that the maximum size of cut may be limited to satisfy code imposed strength requirements under other loading conditions (gravity, wind, etc.) or code imposed drift limitations. These issues will be discussed in Section 5.2.

Addition of Continuity Plates. Prior to 1988, no U.S. building codes contained any specific requirements for continuity plates in seismic-resistant welded moment connections. The *AISC Specification for Structural Steel Buildings* contained the only code provisions governing continuity plates. The 1988 *Uniform Building Code* (UBC) (ICBO 1988) adopted special continuity plate requirements for special moment frames (see Section 2722 (f) 5 of the 1988 UBC) which were more stringent than the basic requirements in the *AISC Specification.* After the Northridge earthquake, the *Interim Guidelines* and *Advisory No. 1* recommended that continuity plates be provided in all cases in new construction of welded moment connections, and that the continuity plate thickness should at least equal the beam flange thickness. Other guidelines for new construction applications of RBS connections have made similar recommendations (Engelhardt et al. 1997). All of the successful tests on RBS connections for new construction (Table 3.1) have employed continuity plates. However, no RBS tests for new construction have omitted continuity plates, so it is unclear under what conditions continuity plates are actually required. The tests on bottom flange RBS modifications of existing pre-Northridge connections (Table 3.4) did not employ continuity plates but still developed plastic rotations of 0.02 radian to 0.025 radian, indicating satisfactory connection performance is possible without continuity plates.

The tendency to always use continuity plates in welded moment connections since the Northridge Earthquake reflects the current lack of appropriate design criteria. Design criteria for continuity plates in seismic-resistant welded steel moment connections in force prior to the Northridge Earthquake have been questioned as being potentially unconservative in some cases. Thus, until new design criteria can be developed, it appears prudent to generally employ continuity plates.

When considering modification of existing pre-Northridge connections, the addition of continuity plates, if they do not already exist, may be very costly. It may require additional slab removal and may also require modifying the connections of beams framing transversely into the column. Considering the uncertainty in continuity plate design criteria, combined with the potentially high cost of adding continuity plates, no general recommendations are made herein with respect to continuity plates when modifying an existing connection using the RBS. The designer, however, should at least consider the possibility of adding continuity plates, if they are not already present, in order to further enhance the performance of the modified connection. Such decisions must be made on a case-by-case basis, considering the construction difficulty and cost involved. As an intermediate measure, the designer may wish to consider adding continuity plates at least in cases where they would have been required by the 1988 UBC (Section 2722 (f) 5). As noted earlier, tests on RBS modifications of existing pre-Northridge connections (Table 3.4) did not employ continuity plates, but still developed on the order of 0.02 radian to 0.025 radian plastic rotation. These test specimens, however, were constructed using columns with relatively thick flanges. Further, these specimens would not have required continuity plates according to the requirements of the 1988 UBC.

Guidelines for sizing and welding continuity plates are provided in the *Seismic Provisions for Structural Steel Buildings* (AISC 1997). The designer is cautioned to avoid welding in the "k-region" of the column. Further information on potential problems in this area can be found in (FEMA 1997, "AISC Advisory" 1997, and "AISC Initiates" 1997).

Modification of Beam Web Connection. Typical pre-Northridge moment connections were constructed with bolted web connections, sometimes with the addition of small supplemental web welds. However, many RBS connections tested for new construction applications since the Northridge Earthquake have employed fully welded web connections. These have been constructed either by welding the beam web directly to the column with a complete joint penetration groove weld, or by the use of a heavy all-welded shear tab. Concerns have been raised in the past that a bolted web connection may not be effective in transferring moment from the beam web to the column due to bolt slippage, even in cases where fully tensioned bolts are used. A fully welded web connection may provide for better force transfer in the web connection, thereby reducing stress levels at the beam flanges and beam flange groove welds, and therefore enhancing connection performance. Although the benefits of a welded web connection are difficult to quantify, experimental data suggests there are clear benefits.

As an additional measure to further enhance connection performance, the designer may wish to consider replacing the existing bolted web connection with a welded web connection. Such measures have been employed in tests involving the repair of damaged pre-Northridge moment connections (FEMA 1997b). In these tests, the existing web bolts were first removed and a heavy web doubler plate, nearly the full depth of the beam web, was attached to the beam web on the side opposite the existing shear tab. This plate was then welded to the face of the column with a groove weld, and was fillet welded to the beam web. A complete description of these web modifications, with detailed welding procedures, can be found in (FEMA 1997b). For modification of an existing undamaged connection, a welded web connection can likely be achieved with a smaller web doubler plate, by directly welding the web to the column via a CJP groove weld, or by other means. As with the other modifications described above, the designer must balance the potential benefits of modifying the web connection against the cost on a case-by-case basis.

Addition of Supplemental Lateral Beam Bracing. For new construction applications of RBS connections, *Advisory No. 1* (FEMA 1996) recommends that lateral bracing be provided near the RBS. This recommendation is based on the concern that removal of flange material at the RBS may promote earlier or more severe lateral torsional buckling of the beam. Examination of RBS test data for new construction (Table 3.1) indicates that some test specimens had additional lateral braces at the RBS. However, the majority of test specimens did not have additional lateral braces with no apparent detrimental effect. Further, tests on RBS modifications of existing connections (Table 3.4) did not provide supplemental lateral bracing. Thus, based on the currently available data, there appears to be no compelling evidence for the need for supplemental lateral bracing at the RBS. Nevertheless, the designer may wish to consider the addition of such bracing as a precaution, as recommended in *Advisory No. 1* (FEMA 1996). If supplemental lateral bracing is provided, the attachment to the beam should be made just beyond the RBS. Attaching a brace within the RBS or in the region between the RBS and the face of the column is not recommended as attachments to the beam in these regions of high inelastic strain may promote a fracture of the beam flange.

5.2 Additional Design Considerations

After designing an RBS modification, the designer must also check that the resulting frame satisfies all appropriate code requirements for strength and stiffness. The strength of the beam at the minimum section of the RBS must satisfy code requirements under all applicable load combinations including gravity, wind, and any other loads appropriate for the structure under consideration. Beam sizes in typical WSMF structures are frequently governed by code specified drift limits. Consequently, even with a reduction in beam strength due to the addition of the RBS, the strength of the modified frame will often be satisfactory for all load combinations. However, if the modified frame fails to satisfy code strength requirements under some load combinations, then the designer should consider an alternative connection modification strategy, such as the addition of haunches or a combination of an RBS with connection reinforcement.

The addition of an RBS modification will reduce the elastic stiffness of a WSMF. This reduction in stiffness, although generally quite small, may affect the ability of the frame to satisfy code specified drift limits. A study by Grubbs (1997) evaluated the reduction in elastic lateral stiffness of WSMFs due to the addition of circular RBS cuts at both the top and bottom flange at each moment connection in a frame. This study showed that over a wide range of WSMF heights and configurations, the average reduction in stiffness for a 50 percent flange reduction was on the order of 5 percent to 7 percent. For a 40 percent flange reduction, the reduction in elastic frame stiffness was on the order of 4 percent to 5 percent. This study did not include stiffness evaluations for cases where the RBS was provided in the bottom flange only. Nevertheless, based on the study by Grubbs, it may be concluded that providing an RBS in the bottom flange only, with a 50 percent flange reduction, is likely to reduce the overall frame stiffness less than about 5 percent. Alternatively, a designer can construct a refined structural model of the modified frame to more accurately assess the reduction in stiffness. In either case, the designer must decide if this reduction in stiffness, and therefore increase in drift, is acceptable.

When evaluating the acceptability of a modified WSMF, a broader issue is the choice of overall design criteria for the modified frame. That is, should the strength and drift evaluations be based on the code under which the frame was originally designed, on the current code, or on some other criteria? The choice of design criteria for seismic rehabilitation is beyond the scope of this document. The reader is referred to FEMA 273 (ATC 33) for additional guidance on this issue.

5.3 Design Example

Description of Existing Frame:

Beam:	W36×150 A36
Column:	W14×426 A572 Gr. 50

Centerline dimensions:

- story height: 12 ft
- bay width: 30 ft

Factored gravity load on moment frame beams: 0.6 kips/ft (0.05 kips/in.)

Existing connection:

- welded flange—bolted web connection to column flange—interior connection
- beam flange groove welds: E70T-4 FCAW with steel backing and weld tabs left in place
- beam web connection:
  - 9-1" A325 bolts
  - 5/8 in. × 5 in. × 27 ½ in. shear tab, connected to column with 5/16 in. fillet welds
  - no supplemental web welds between shear tab and beam web
- no continuity plates
- no doubler plates

Building constructed in early 1980's

Section Properties:

W36×150: $d = 35.85$ in.
$b_f = 11.975$ in.
$t_f = 0.94$ in.
$t_w = 0.625$ in.
$Z = 581$ in.^{3}

W14×426: $d = 18.67$ in.
$b_f = 16.695$ in.
$t_f = 3.305$ in.
$t_w = 1.875$ in.
$Z = 869$ in.^{3}

Expected Yield Stress of Beam Flange

W36×150 beam was specified as A36. No testing was conducted on steel samples from building and no CMTRs are available. Therefore, estimate F_{ye} based on Eq. 4.1 and Table 4.1. Thus:

$$F_{ye} = R_y F_y = 1.3 \times 36 = 47 \text{ ksi}$$

Connection Modification Design:

- Modification: Provide circular cut RBS in beam bottom flange only
- Preliminary dimensions of RBS cut:

$a \approx (0.5 \text{ to } 0.75)\, b_f \approx 6$ in. to 9 in.: Try: $a = 6$ in.
$b \approx (0.65 \text{ to } 0.85)\, d \approx 23$ in. to 30 in.: Try: $b = 27$ in.
$c \approx 0.25\, b_f \approx 3.0$ in.: Try: $c = 3$ in.

- Compute Z_{RBS} (Eq. 5.5):

$$Z_{RBS} = Z_b - \frac{(ct_f)^2}{t_w} - ct_f(d - t_f)$$

$$= 581 - \frac{(3.0 \times .94)^2}{0.625} - (3.0 \times 0.94)(35.85 \times 0.94)$$

$$= 470 \text{ in.}^3$$

- Compute s_c and L':

$$s_c = a + (b/2) = 6 + (27/2) = 19.5 \text{ in.}$$

$$L' = (\text{bay width}) - (\text{col. depth}) - 2\, s_c = 360 \text{ in.} - 19 \text{ in.} - 2 \times 19.5 \text{ in.} = 302 \text{ in.}$$

- Compute η (Eq. 5.8):

$$\eta = \left(1 + \frac{2s_c}{L'}\right)1.1\frac{Z_{RBS}}{Z_b} + \frac{wL's_c}{2Z_bF_{ye}}$$

$$= \left(1 + \frac{2 \times 19.5}{302}\right) \times 1.1 \times \frac{470}{581} + \frac{0.05 \times 302 \times 19.5}{2 \times 581 \times 47}$$

$$= 1.01$$

- Check that $\eta \leq 1.05$: <u>OK</u>

Preliminary RBS dimensions are OK. Use:

$$a = 6 \text{ in.}$$

$$b = 27 \text{ in.}$$

$$c = 3 \text{ in.}$$

- Beam flange groove weld modifications:
  - Remove existing weld metal;
  - Reweld using an electrode with $F_{EXX} = 70$ ksi and minimum specified CVN of 20 ft-lb at −20°F;
  - Remove weld tabs at top and bottom flanges;
  - Remove bottom flange steel backing and provide 5/16 in. fillet weld at base of groove weld after the root is cleaned and inspected;
  - Leave top steel backing in place; provide 5/16 in. fillet weld between steel backing and column flange.
- Column-Beam Moment Ratio (Section 4.3.3):

$$M_{pd} = 1.1 Z_{RBS} F_{ye} = 1.1 \times 470 \times 47$$

$$= 24{,}300 \text{ kip-in.}$$

$$V_{pd} = (2\, M_{pd})/L' + (wL'/2)$$

$$= (2 \times 24300)/302 + (0.05 \times 302/2)$$

$$= 168 \text{ kips}$$

$$V_c = \frac{\sum\left[M_{pd} + V_{pd}(L - L')/2\right]}{h_b + d_p + h_t}$$

$$= \frac{2 \times \left[24{,}300 + 168(360 - 302)/2\right]}{144}$$

$$= 405 \text{ kips}$$

$$M_{ct} = V_c h_c = 405(144 - 36)/2 = 21{,}870 \text{ kip-in.}$$

$$M_{cb} = V_c h_b = 405(144 - 36)/2 = 21{,}870 \text{ kip-in.}$$

$$\frac{\sum Z_c(F_{yc} - P/A_c)}{\sum M_c} = \frac{2 \times 869(50 - 10)}{2 \times 21{,}870}$$

$$= 1.59 > 1.0 \quad \text{OK}$$

(Note: calculation assumes $P/A_c = 10$ ksi)

- Check column panel zone:

Check column panel zone according to the recommendations of Section 7.5.2.6 of FEMA 267A.

Requirement:

The panel zone shear force due to $0.8\Sigma M_F$ shall not exceed the panel zone shear strength, given by:

$$V = 0.55 F_{yc} d_c t \left[1 + \frac{3 b_c t_{cf}^2}{d_b d_c t}\right]$$

Compute M_f (Eq. 5.4):

$$M_f = \left[1 + \frac{2 s_c}{L'}\right] M_{pd} + \frac{w L' s_c}{2}$$

$$= \left[1 + \frac{2 \times 19.5}{302}\right] \times 24{,}300 + \frac{0.05 \times 302 \times 19.5}{2}$$

$$= 27{,}585 \text{ kip-in.}$$

Compute panel zone shear force due to $0.8 \sum M_F$:

$$V = \frac{0.8 \sum M_F}{0.95 d_b} - 0.8 V_c$$

$$= \frac{0.8 \times 2 \times 27{,}585}{0.95 \times 35.85} - 0.8 \times 405 = 972 \text{ kips}$$

Compute panel zone shear strength:

$$V = 0.55 F_{yc} d_c t \left[1 + \frac{3 b_c t_{cf}^2}{d_b d_c t}\right]$$

$$= 0.55 \times 50 \times 18.67$$

$$\times 1.875 \left[1 + \frac{3 \times 16.7 \times 3.04^2}{35.85 \times 18.67 \times 1.875}\right]$$

$$= 1{,}318 \text{ kips} > 972 \text{ kips}$$

$\therefore$ *No doubler plates required*

- Continuity plates:

Existing connection was not provided with continuity plates. Provide continuity plates only if required according to criteria in 1988 UBC (Section 2722 (f) 5).

Continuity plates required if:

$$t_{cf} < 0.4 \sqrt{\frac{P_{bf}}{F_{yc}}}$$

where

$$P_{bf} = 1.8 (b t_f) F_{yb}$$

Note: Use $F_{yb} = F_{ye} = 47$ ksi

$$P_{bf} = 1.8 \times 11.975 \times 0.94 \times 47$$

$$= 953 \text{ kips}$$

$$0.4 \sqrt{\frac{P_{bf}}{F_{yc}}} = 0.4 \sqrt{\frac{953}{50}} = 1.74 \text{ in.}$$

$$t_{cf} = 3.04 \text{ in.} > 1.74 \text{ in.}$$

$\therefore$ *No continuity plates required*

Also check other column stiffener requirements per Chapter K of the LRFD Specification for Structural Steel Buildings.

Per Chapter K of LRFD Specification: no continuity plates req'd.

$\therefore$ *Do not add continuity plates to the modified connection*

- Effect of RBS on building drift:

As noted in Section 5.2, the addition of a bottom flange RBS with 50 percent flange removal, if provided at every moment connection in the frame, is expected to increase elastic lateral drift by less than 5 percent. This small increase in drift is considered acceptable for this building.

Chapter 6
DESIGN OF WELDED HAUNCH MODIFICATION

This chapter deals with the modification of pre-Northridge steel moment frame connections using a welded haunch. Only the triangular haunch, welded to the beam bottom flange, is considered. With minor modifications, the procedure presented here is also applicable to the addition of haunches to both top and bottom flanges. The welded triangular haunch features a strut action, allowing for the beam shear to be transferred to the column via the haunch flange. Other types of welded haunch (e.g., straight haunch where only the haunch web is welded to the beam) which do not feature such a strut action are beyond the scope of this section.

6.1 Recommended Design Procedure

The effectiveness of the welded haunch in enhancing the seismic performance of pre-Northridge moment connections has been demonstrated by full-scale testing. The presence of a welded haunch dramatically changes the beam shear force transfer mechanism. Both theoretical studies and experimental results have shown that the majority of the beam shear is transferred to the column through the haunch flange rather than through the beam flange groove welds (Goel and Stojadinovic 1997). This strut action also alters the moment distribution of the beam in the haunch region. The force demand in the existing bottom flange groove weld is significantly reduced, and the force demand in the existing top flange groove weld can be reduced to a reasonable level. The addition of a haunch enlarges the column panel zone and thereby increases the strength of the panel zone. Nevertheless, using a haunch to force the plastic hinge to occur away from the column face would make the strong column-weak beam condition more difficult to meet. Detailed information on the theoretical background and design procedure for the welded haunch can be found in (Yu et al. 1997). The following issues are addressed in this section:

- Force transfer mechanism,
- Flexural stress at beam top and bottom flanges, and
- Dual panel zone behavior.

6.7.7 *Structural Behavior and Design Considerations*

To begin a trial design, one must first select a haunch geometry, that is, the length of the haunch and the angle of the haunch flange with respect to the beam axis. In the tests conducted to date, these two parameters for the majority of test specimens have varied only to a small extent. It seems prudent to remain within the limits of experimental evidence and, therefore, based on the data presented in Tables 3.2 and 3.5, it is suggested that the length of the haunch, a, and angle, θ, (see Figure 6.1) be taken as

$$a = (0.5 \text{ to } 0.6)d \tag{6.1}$$

$$\theta = 30^\circ \pm 5^\circ \tag{6.2}$$

The designer may want to check the value of $b\,(= a \tan\theta)$ to ensure that the haunch does not interfere with the ceiling.

Design of a welded haunch is based on the moment and shear that develop at the tip of the haunch (see Table 4.2). The design moment at the plastic hinge, M_{pd}, is given by Eq. 4.2, in which the factor α is intended to account for strain hardening. To obtain the value of α, available test data for eight tests of the single haunch modification were analyzed and the results are shown in Figure 6.2. Two plots are presented for two different beam sizes: W30×99 and W36×150. The abscissa represents the Story Drift Ratio (SDR), and the ordinate is the normalized moment at the haunch tip. The normalization is based on the actual plastic moment of the beam, where the beam flange yield stresses were obtained from tension coupon tests. It

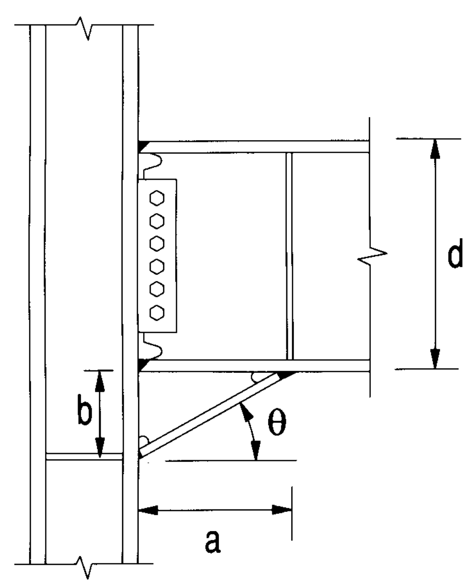


Figure 6.1 Haunch Geometry

is observed that the beam maximum moment can be slightly larger than the actual plastic moment, and using a value of 1.1 for α in Eq. 4.2 is reasonable for design purposes. Thus, M_{pd} may be computed as follows:

$$M_{pd} = 1.1 Z_b F_{yef} \tag{6.3}$$

where

Z_b = plastic section modulus of the beam, and
F_{yef} = expected yield stress of the beam flanges as determined in Section 4.1.

Once M_{pd} is determined, the corresponding beam shear, V_{pd}, can be computed as follows:

$$V_{pd} = \frac{2M_{pd}}{L'} + V_G \tag{6.4}$$

where

L' = beam span between plastic hinges (see Figure 4.1), and
V_G = beam shear at the plastic hinge location produced by gravity load in beam span L'.

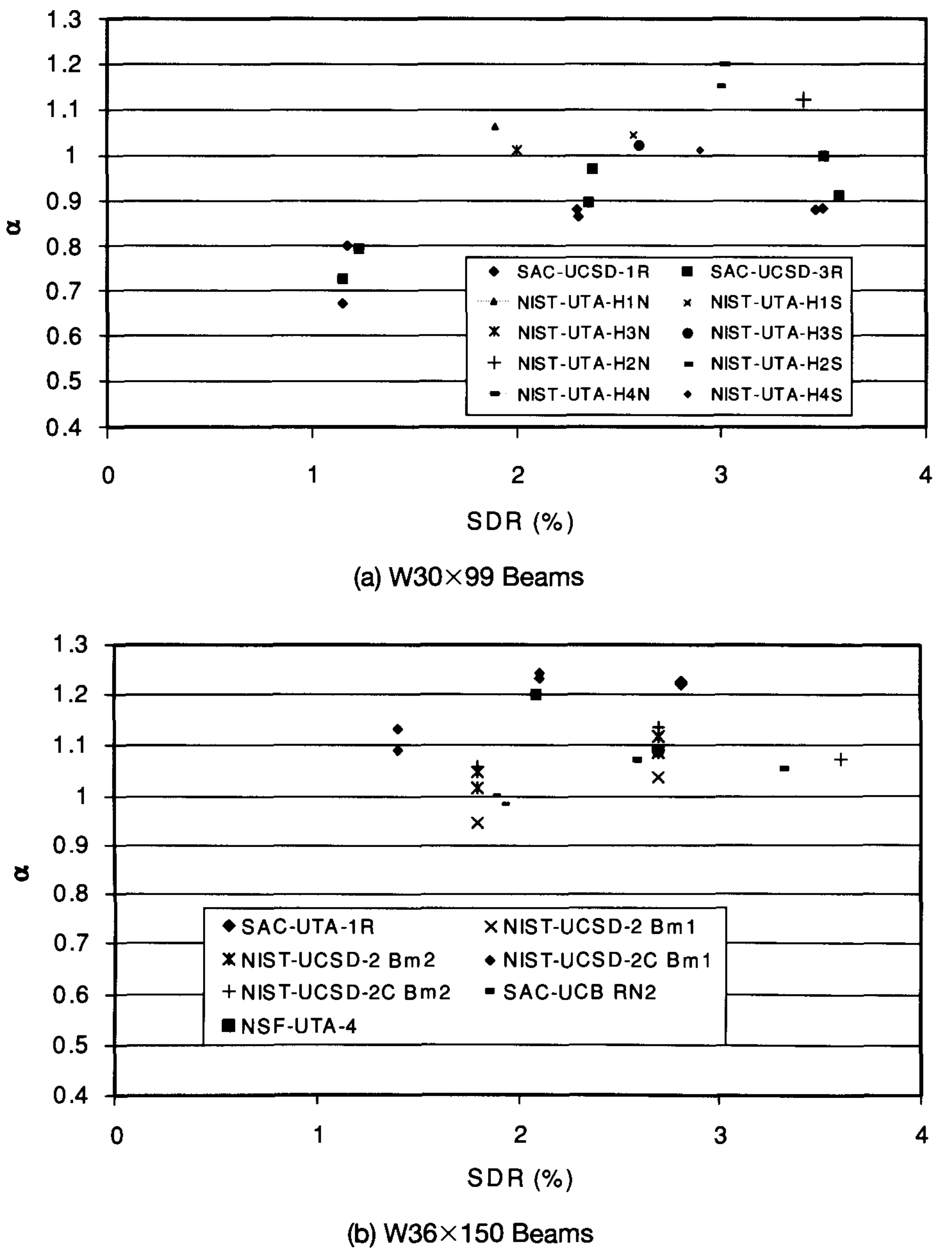


Figure 6.2 Story Drift Ratio (SDR) versus Moment Ratio

6.7.2 *Simplified Haunch Connection Model and Determination of Haunch Flange Force*

For economic reasons, it is desirable not to modify the existing beam flange groove welds. NIST/AISC test results have shown that brittle fracture of the beam top flange groove weld did not occur when the composite slab was present even though strain gage measurements indicated that the beam top flange not only yielded but also strain-hardened. One reason that the top flange groove weld was able to tolerate higher tensile stresses might be that the beam shear was primarily transmitted through the haunch, not the beam flange groove welds. Based on strain gage measurements, the beam top flange strain near the column face was found to approach 20 to 30 times the yield strain. Since the yield stress of the beam flange for the NIST/AISC specimens was about 50 ksi, the tensile stress in the beam top flange and its groove weld (with E7XT-X electrode) likely exceeded 55 ksi. In this Design Guide, it is recommended that the allowable stress for the existing groove weld be taken as $0.8F_{EXX}$, where F_{EXX} is the strength of the weld metal. For a 70 ksi tensile strength electrode, this requirement would limit the allowable stress in the groove weld to 0.8 X 70 = 56 ksi.

For modification design, both experimental evidence and finite element analysis results have shown that classical beam theory (i.e., Mc/I where I is the moment of inertia of the section including both the beam and haunch) cannot predict reliably the distribution of beam flexural stresses at the column face. A procedure that was developed for estimating the flexural stress distribution is described herein (Yu et al. 1997). Figure 6.3 shows the model for a haunch connection, where the haunch is idealized as a spring and the finite depth of the beam is also considered. At the haunch tip, the amount of beam shear force that is transferred to the haunch flange is a function of the axial stiffness of the haunch flange. It can be shown that the contribution of the haunch web to the axial stiffness of the haunch flange is minor and can be ignored.

Let the vertical component of haunch flange axial force be βV_{pd}, where β remains to be established (see Figure 6.4(a)); the horizontal component of haunch flange axial force is then equal to $\beta V_{pd}/\tan\theta$. Such a horizontal force component together with an eccentricity of $d/2$ due to the finite depth of the beam produces a tensile force and concentrated moment to the beam in the haunch region (see Figure 6.4(b)). The tensile force would increase the tensile stress of the beam top flange at the column face. This tensile stress, however, is always less in magnitude than the compressive stress produced by the concentrated moment. Figure 6.4(c) shows the beneficial effect of this concentrated moment in reducing the beam moment in the haunch region. If β is equal to one, i.e., all the beam shear (V_{pd}) is transmitted to the haunch flange, the beam shear in the haunch region vanishes, and the beam moment is constant in that region. When β is larger than one, the beam shear in the haunch region is reversed in direction as compared to that outside the haunch region. Since beam shear is the slope of the moment diagram, such a reverse shear further reduces the beam moment (and, hence, tensile stress in the groove weld) at the column face as shown in Figure 6.4(c).

Since the majority of the beam shear is transferred through the haunch flange to the column, for design

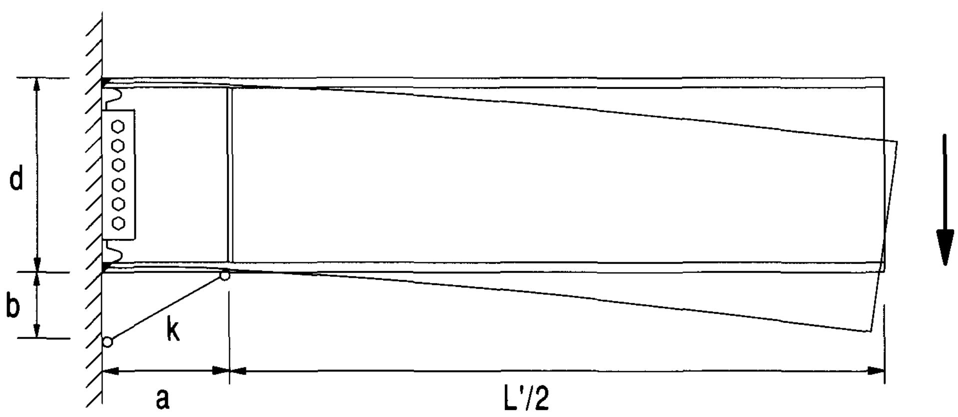
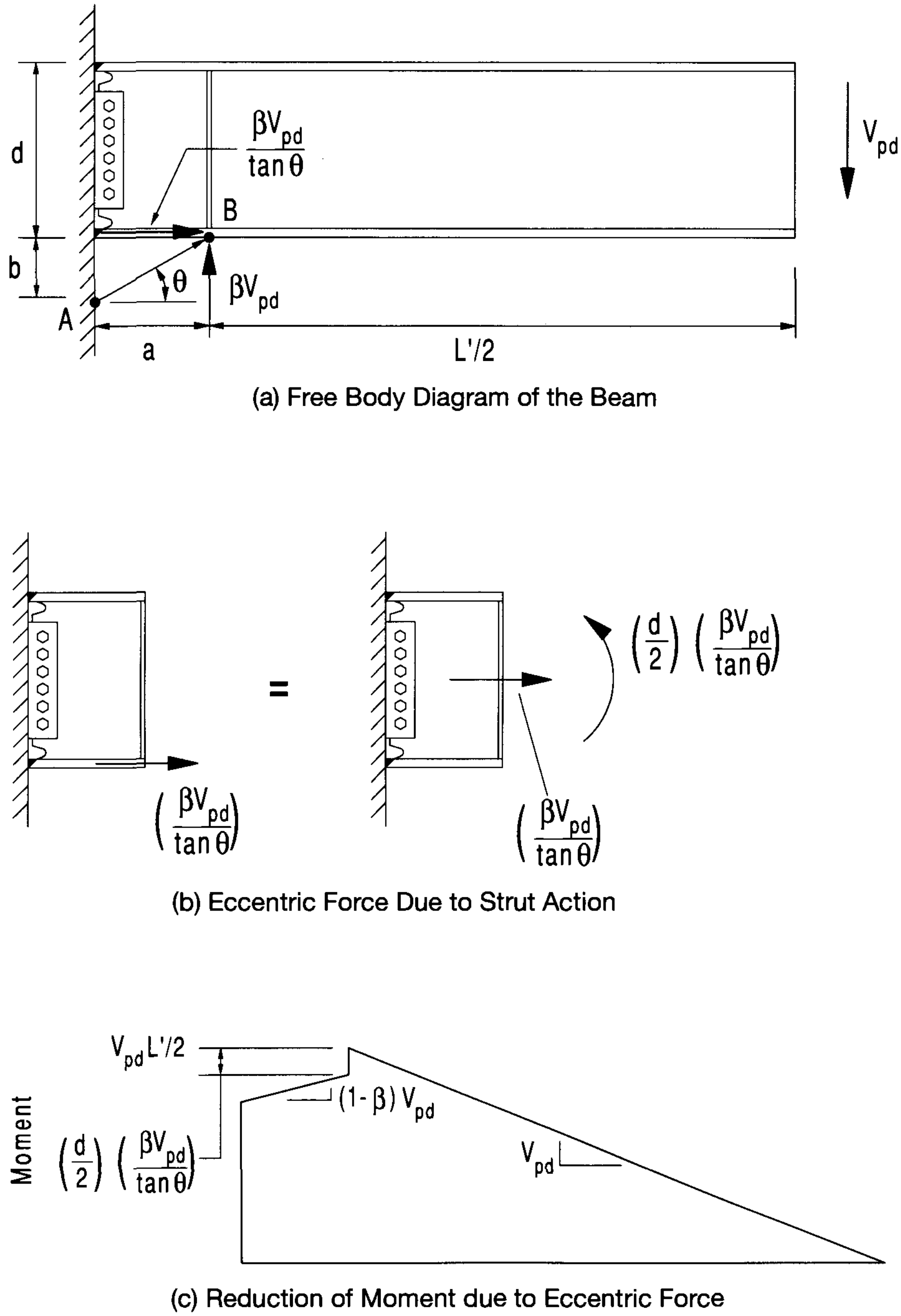


Figure 6.3 Simplified Model of Haunch Connection



(a) Free Body Diagram of the Beam

(b) Eccentric Force Due to Strut Action

(c) Reduction of Moment due to Eccentric Force

Figure 6.4 Free Body and Moment Diagrams of Haunch Reinforced Beam purposes, the beam top flange stress at the column face can be calculated by beam theory as follows:

$$f_w = \frac{V_{pd}(L'/2+a)}{I_b}\left(\frac{d}{2}\right) - \frac{\beta V_{pd} a}{I_b}\left(\frac{d}{2}\right) - \frac{(\beta V_{pd}/\tan\theta)(d/2)}{I_b}\left(\frac{d}{2}\right) + \frac{\beta V_{pd}/\tan\theta}{A_b}$$

$$= \frac{V_{pd}L'/2 + V_{pd}(1-\beta)a}{I_b}\left(\frac{d}{2}\right) - \frac{(\beta V_{pd}/\tan\theta)}{I_b}\left(\frac{d^2}{4} - \frac{I_b}{A_b}\right) \tag{6.5}$$

where I_b and A_b are the moment inertia and area of the beam section, respectively. Substituting the bending moment at the haunch tip ($= V_{pd}L'/2$) by M_{pd}, the above equation can be re-written as follows:

$$f_{wt} = \frac{M_{pd} + V_{pd}(1-\beta)a}{I_b}\left(\frac{d}{2}\right) - \frac{\beta V_{pd}/\tan\theta}{I_b}\left(\frac{d^2}{4} - \frac{I_b}{A_b}\right) \tag{6.6}$$

The minimum value of β can be determined by solving Eq. 6.6 and equating f_{wt} to the allowable stress, F_w:

$$\beta_{\min} = \frac{(M_{pd} + V_{pd}a)/S_x - F_w}{\frac{V_{pd}a}{S_x} + \frac{V_{pd}}{I_b\tan\theta}\left(\frac{d^2}{4} - \frac{I_b}{A_b}\right)} \tag{6.7}$$

The haunch flange axial force, P_{hf}, is equal to $\beta V_{pd}/\sin\theta$ and once the minimum value of βV_{pd} is determined, the haunch flange can be sized as follows:

$$A_{hf} \geq \frac{P_{hf}}{\phi F_y} = \frac{\beta V_{pd}}{\phi F_y \sin\theta} \tag{6.8}$$

where

A_{hf} = haunch flange area = $b_{hf}t_{hf}$,
b_{hf} = haunch flange width,
t_{hf} = haunch flange thickness,
ϕ = 0.9, and
F_y = minimum specified yield stress of haunch flange.

The haunch flange should satisfy the following width-thickness requirement for a compact section:

$$\frac{b_{hf}}{2t_{hf}} \leq \frac{52}{\sqrt{F_y}} \tag{6.9}$$

In addition to satisfying the strength requirement (Eq. 6.8) and the stability requirement (Eq. 6.9), it is necessary to check the axial stiffness of the designed haunch flange to ensure that the minimum vertical component ($\beta_{min}V_{pd}$) of the reaction, as computed from Eq. 6.7, can be developed by the haunch flange. This vertical component can be computed by considering the deformation compatibility between the beam and haunch. See Appendix A.1 for detailed derivations. The resulting β factor can be expressed as follows:

$$\beta = \left(\frac{b}{a}\right)\frac{3L'd + 3ad + 3bL' + 4ab}{3d^2 + 6bd + 4b^2 + \frac{12I_b}{A_b} + \frac{12I_b}{A_{hf}\cos^3\theta}} \tag{6.10}$$

The actual β value thus computed cannot be less than β_{min}. If the haunch flange is conservatively designed, the actual β value will be significantly larger than the β_{min}. value. In such cases, the designer may consider reducing the haunch flange area.

Based on Figure 6.5(b), the beam bottom flange force, P_{bf}, to the left of the haunch tip (point B) is much smaller than that in the top flange due to the contribution of the horizontal component of the haunch flange force (see Figure 6.4(b)). To compute the maximum tensile stress in the beam bottom flange groove weld when the beam is subjected to positive bending, i.e., when V_{pd} in Figure 6.4(a) acts upward, the following equation can be derived with minor modifications to Eq. 6.5:

$$f_{wb} = \frac{V_{pd}(L'/2+a)}{I_b}\left(\frac{d}{2}\right) - \frac{\beta V_{pd} a}{I_b}\left(\frac{d}{2}\right) - \frac{(\beta V_{pd}/\tan\theta)(d/2)}{I_b}\left(\frac{d}{2}\right) - \frac{\beta V_{pd}/\tan\theta}{A_b}$$

$$= \frac{V_{pd}L'/2 + V_{pd}(1-\beta)a}{I_b}\left(\frac{d}{2}\right) - \frac{(\beta V_{pd}/\tan\theta)}{I_b}\left(\frac{d^2}{4} + \frac{I_b}{A_b}\right) \tag{6.11}$$

Note that the contribution of the haunch web is excluded in the force equilibrium in Figure 6.4(b) because its stiffness in the haunch flange direction is small. But the haunch web plays an important role in providing stability to the haunch flange. For design purposes, it is suggested that the thickness of the haunch web satisfy the following requirement:

$$\frac{a \sin\theta}{t_{hw}} \leq \frac{260}{\sqrt{F_y}} \tag{6.12}$$

The above requirement is established by treating the haunch as half of a wide-flange beam section whose depth is twice the distance $a \sin\theta$ (see Figure 6.6) and limiting the width-thickness ratio, $2a \sin\theta / t_{hw}$, to $520/\sqrt{F_y}$ per the AISC *Seismic Provisions for Structural Steel Buildings* (1997).

6.1.3 Haunch Web Shear

Although the haunch web does not participate in the force equilibrium at the haunch tip, shear stresses do result in the haunch web due to the deformation compatibility between the haunch flange and haunch web (see Figure 6.7). Treating the haunch web as a first-order triangular finite element, the average shear stress in the haunch web can be derived as follows:

$$\tau_{hw} = \frac{aV_{pd}}{2(1+\nu)I_b}\left(\frac{L'}{2} - \frac{\beta}{\tan\theta}\left(\frac{d}{2}\right) + \frac{(1-\beta)a}{3}\right) \tag{6.13}$$

where ν (= 0.3) is Poisson's Ratio. (See Appendix A.2 for detailed derivations.) The shear stress computed from Eq. 6.13 above should not exceed the allowable shear strength:

$$\tau_{hw} \leq \phi_v(0.6F_y) \tag{6.14}$$

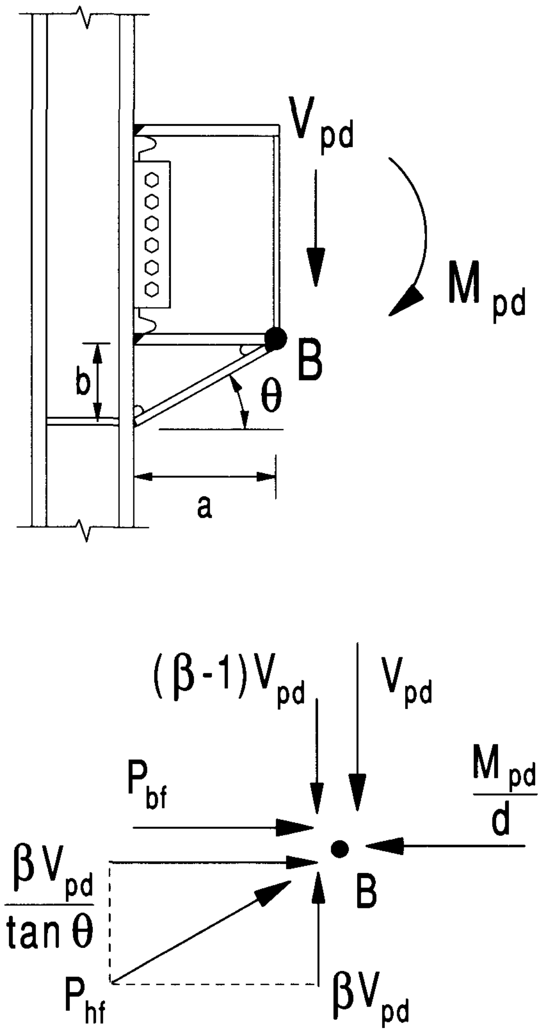


Figure 6.5 Force Equilibrium at Haunch Tip

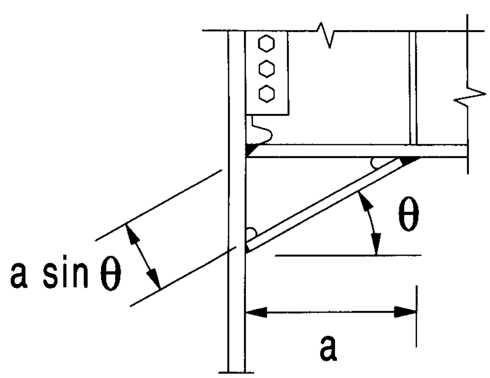


Figure 6.6 Haunch Web

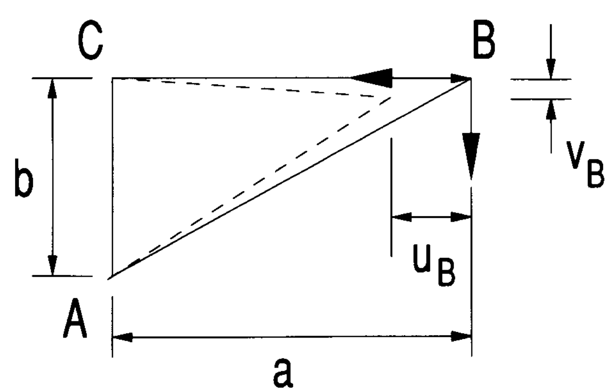


Figure 6.7 Deformation ofHaunch Web

where

$\phi_v = 0.9.$

From the slope of the beam moment diagram (i.e., beam shear) in Figure 6.4(c), it is observed that a shear of magnitude $(\beta - 1)V_{pd}$ in the direction of the beam shear outside the haunch is developed in the haunch region; the direction of this beam shear is opposite to that developed outside the haunch region if β is larger than one. Therefore, the shear force in the beam web is

$$V_{bw} = (1 - \beta)V_{pd} \tag{6.15}$$

In general, the value of V_{bw} is significantly less than that of V_{pd}, indicating that the existing beam flange groove welds and the beam web connection only need to transfer a small amount of shear force. If the value of V_{bw} is negative, it means that the direction of the beam shear in the haunch region is reversed. If the existing beam web connection does not have a sufficient capacity to resist V_{bw}, additional welding of the beam web may be required to increase the shear capacity.

Check Dual Panel Zone Shear Strength. The presence of a haunch also creates an enlarged (or "dual") panel zone. Usually the increase in shear strength is larger than the increase in shear demand. If desired, the designer can use the procedure developed by Lee and Uang (1995) to compute the shear strength of the dual panel zone.

6.1.4 Design Procedure

The design procedure for a welded haunch is summarized as follows:

STEP 1 Select a preliminary haunch geometry using Eqs. 6.1 and 6.2.

STEP 2 Compute the beam design plastic moment (Eq. 6.3) and beam shear (Eq. 6.4).

STEP 3 Check for strong column-weak beam condition (see Section 4.3.3).

STEP 4 Compute the required β_{min} value using Eq. 6.7 to limit the top flange groove weld stress to an allowable value $(0.8F_{EXX})$.

STEP 5 Select a haunch section satisfying Eq. 6.8 and check for compact section requirements using Eq. 6.9.

STEP 6 Use Eq. 6.10 to compute the actual β value and check if the haunch flange has a sufficient stiffness to develop the required β. Increase the haunch flange area or modify the haunch geometry if β is less than β_{min}. The designer may consider reducing the haunch flange area if β is significantly larger than β_{min}.

STEP 7 Use Eqs. 6.13 and 6.14 to check the shear capacity of the haunch web. Use Eq. 6.15 to check the shear capacity of the beam web bolted connection.

6.2 Recommended Detailing Provisions

6.2.7 *Design Weld*

Groove weld with a specified Charpy V-Notch toughness of 20 ft-lb at −20°F should be used to connect the haunch flange to both the column and beam flanges. Connections between the haunch web and both the column and beam flanges should have sufficient strength per unit length to resist the following shear force:

$$v_{hw} = \tau_{hw} t_{hw} \tag{6.16}$$

A sample welded haunch detail is shown in Figure 6.8.

6.2.2 *Design Stiffeners*

Since the haunch flange exerts a concentrated force on the beam, it is suggested that a pair of transverse stiffeners be added to the beam web at the location where the haunch flange intersects the beam. At a minimum, the stiffeners should extend at least one half the beam depth and the width-to-thickness of each stiffener should be limited to $95/\sqrt{F_y}$. Such a measure would ensure that the vertical reaction (βV_{pd}) at the haunch tip would not be reduced by the flexibility of the beam web. Using full-depth stiffeners is desirable because their presence increases the likelihood that local buckling of the beam top flange would

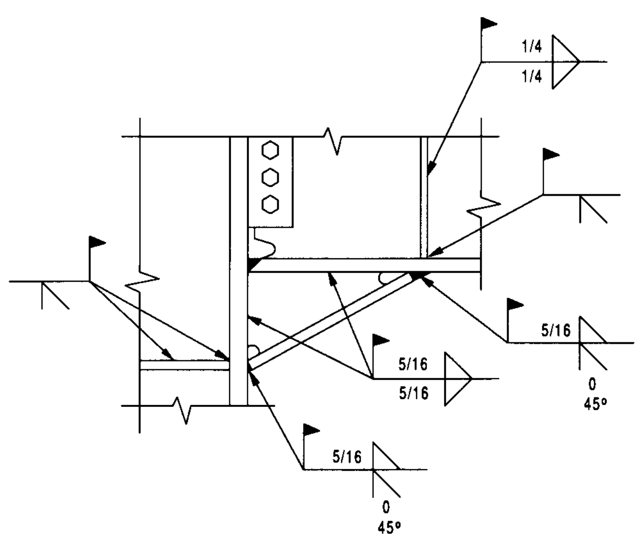


Figure 6.8 Typical Haunch Weld Details

occur outside the haunch region, not next to the column face at the groove welds.

The beam web together with a pair of transverse stiffeners should also be checked per Chapter K of the AISC *LRFD Specification* (1993) for local flange bending, local web yielding, and web crippling to ensure the strength is sufficient to resist a concentrated force of βV_{pd}. When full-depth stiffeners are used, Section K1.9 in Chapter K requires that the stiffened beam web be designed as an axially compressed member with an effective length of $0.75h$ (h = clear distance between beam flanges less the fillet radius), a cross section composed of two stiffeners and a strip of the beam web having a width of 12 times the beam web thickness. Transverse stiffeners should be welded to the bottom flange to develop the strength of the stiffeners. The weld connecting transverse stiffeners to the web should be sized to transmit the unbalanced force in the stiffener to the web.

6.2.3 Continuity Plates

Whenever possible, it is desirable to add a pair of continuity plates at the beam top flange level to reduce the stress concentration in the groove weld. A pair of continuity plates should always be added at the location where the haunch flange intersects the column. The continuity plates, designed for a concentrated force of $\beta V_{pd}/\tan\theta$, should satisfy the requirements in Chapter K of the LRFD Specification.

6.3 Design Example

Description of Existing Frame:

Beam: W36×150, A36 steel
Column: W14×426, A572 Gr. 50 steel

Centerline Dimensions:

- story height: H_c = 12 ft
- bay width: L = 30 ft

Existing Interior Moment Connection:

- welded flange-bolted web connection to column flange
- beam flange groove welds: E70T-4 FCAW with steel backing and weld tab left in place
- beam web connection:
  - nine 1"-diameter A325 high strength bolts
  - 5/8-in. × 5-in. × 27½-in. shear tab connected to column with 5/16-in. fillet welds
  - no supplemental web welds between shear tab and beam web
- no continuity plates
- no doubler plates

Building constructed in early 1980's.

Member Section Properties:

W36×150:
$$d = 36 \text{ in.}$$
$$b_{bf} = 12 \text{ in.}$$
$$t_{bf} = 0.94 \text{ in.}$$
$$t_{bw} = 0.625 \text{ in.}$$
$$A_b = 44.3 \text{ in.}^2$$
$$I_x = 9040 \text{ in.}^4$$
$$Z_{bx} = 581 \text{ in.}^3$$

W14×426:
$$d_c = 18.67 \text{ in.}$$
$$t_{cw} = 1.875 \text{ in.}$$
$$t_{cf} = 3.305 \text{ in.}$$
$$b_{cf} = 16.695 \text{ in.}$$
$$Z_{cx} = 869 \text{ in.}^3$$

Connection Modification Design:

Consider a uniformly distributed gravity load (w_g = 0.6 kips/ft) for the beam. Assume a column axial stress (f_a) of 10 ksi.

$$L' = 360 - 18.67 - 2 \times 18 = 305.3 \text{ in.}$$

STEP 1: Preliminary dimension of haunch.

$$a = (0.5 \text{ to } 0.6)d_b \qquad \text{Choose } a = 18 \text{ in.}$$
$$\theta = 30^\circ \pm 5^\circ \qquad \text{Choose } \theta = 31^\circ$$
$$b = a\tan\theta = 10.8 \text{ in.}$$

STEP 2: Determine beam probable plastic moment, M_{pd}, and beam shear, V_{pd}.

$$F_{yef} = 1.3F_y = 1.3(36) = 46.8 \text{ ksi}$$

$$M_{pd} = \alpha Z_{bx} F_{yef} = 1.1 \times 581 \times 46.8 = 29{,}910 \text{ kip-in.}$$

$$L' = 360 - 18.67 - 2 \times 18 = 305.3 \text{ in.}$$

$$V_{pd} = 2M_{pd}/L' + w_g \times L'/2 = 2 \times 29{,}910/305.3 + 0.05 \times 305.3/2 = 203.5 \text{ kips}$$

STEP 3: Check for strong column-weak beam condition.

$$\sum M_c = \left[2M_{pd} + V_{pd}(L - L')\right]\left(\frac{H_c - d_b}{H_c}\right)$$
$$= [2 \times 29{,}910 + 203.5 \times (360 - 305.3)]$$
$$\times (144 - 46.65)/144 = 47{,}966 \text{ kip-in.}$$

$$\frac{\sum Z_c(F_{yc} - f_a)}{\sum M_c} = \frac{2 \times 869 \times (50 - 10)}{47{,}966}$$
$$= 1.45 > 1.0 \qquad \text{OK}$$

STEP 4: Determine required minimum β.

Use Equation 6.7 to compute the required β, where F_w = 0.8, F_{EXX} =56 ksi.

$$\beta = \frac{(M_{pd} + V_{pd}a)/S_x - F_w}{\dfrac{V_{pd}a}{S_s} + \dfrac{V_{pd}}{I_b \tan\theta}\left(\dfrac{d^2}{4} - \dfrac{I_b}{A_b}\right)}$$

$$= \frac{(29{,}910 + 203.5 \times 18)/504 - 56}{\dfrac{203.5 \times 18}{504} + \dfrac{203.5}{9040 \times 0.601}\left(\dfrac{35.85^2}{4} - \dfrac{9040}{44.2}\right)}$$

$$= 0.91$$

STEP 5: Size haunch flange.

Use Equation 6.8 to size the haunch flange for strength:

$$A_{hf} \geq \frac{P_{hf}}{\phi F_y} = \frac{\beta V_{pd}}{\phi F_y \sin\theta}$$

$$= \frac{0.91 \times 203.5}{0.9 \times 50 \times 0.515} = 7.99 \text{ in.}^2$$

Select W18×86 (A572 Gr. 50 steel), which provides a haunch flange area of 8.54 in.^{2} (= $b_{hf} \times t_{hf}$ =11.2× 0.77). Check Eq. 6.9 for the compact section requirement:

$$\frac{b_{hf}}{2t_{hf}} = \frac{11.09}{2(0.77)} = 7.2 \leq \frac{52}{\sqrt{F_y}} = 7.35 \qquad \text{OK}$$

STEP 6: Verify the β value for stiffness requirement.

Compute actual β using Equation 6.10 for the haunch flange stiffness requirement:

$$\beta = \frac{b}{a} \times \left(\frac{3L'd + 3ad + 3bL' + 4ab}{3d^2 + 6bd + 4b^2 + \dfrac{12I_b}{A_b} + \dfrac{12I_b}{A_{hf}\cos^3\theta}}\right)$$

$$= \frac{10.8}{18} \times \left[\frac{(3 \times 305.3 \times 35.85 + 3 \times 18 \times 35.85 + 3 \times 10.8 \times 305.3 + 4 \times 18 \times 10.8)}{\left(3 \times 35.85^2 + 6 \times 10.8 \times 35.85 + 4 \times 10.8^2 + \dfrac{12 \times 9040}{44.2} + \dfrac{12 \times 9040}{8.54 \times (0.857)^3}\right)}\right]$$

$$= 0.93 > \beta_{min} \qquad \text{OK}$$

The haunch thus sized would ensure that the tensile stress in the top flange groove weld is limited to the allowable stress, F_w = 56 ksi. The tensile stress in the top flange groove weld can be computed from Equation 6.6:

$$f_w = \frac{M_{pd} + V_{pd}(1-\beta)a}{I_b}\left(\frac{d}{2}\right) - \frac{\beta V_{pd}/\tan\theta}{I_b}\left(\frac{d^2}{4} - \frac{I_b}{A_b}\right)$$

$$= \frac{29{,}910 + 203.5 \times (1 - 0.93) \times 18}{9040} \times \left(\frac{35.85}{2}\right) - \frac{0.93 \times 203.5/0.60}{9040} \times \left(\frac{35.85^2}{4} - \frac{9040}{44.2}\right)$$

$$= 55.7 \text{ ksi} < 56 \text{ ksi} \qquad \text{OK}$$

The haunch flange axial stress is

$$\frac{\beta V_{pd}}{A_{hf}\sin\theta} = \frac{0.93 \times 203.5}{8.54 \times 0.515} = 43.0 \text{ ksi} < \phi F_y = 45 \text{ ksi, where } \phi = 0.9 \qquad \text{OK}$$

Therefore, the selected haunch flange provides adequate stiffness and strength. The maximum tensile stress in the groove weld of the beam bottom flange can be computed from Eq. 6.11.

$$f_{wb} = \frac{V_{pd}L'/2 + V_{pd}(1-\beta)a}{I_b}\left(\frac{d}{2}\right) - \frac{(\beta V_{pd}/\tan\theta)}{I_b}\left(\frac{d^2}{4} + \frac{I_b}{A_b}\right)$$

$$= \frac{29{,}910 + 203.5 \times (1 - 0.93) \times 18}{9040}\left(\frac{35.85}{2}\right) - \frac{0.93 \times 203.5/0.60}{9040}\left(\frac{35.85^2}{4} + \frac{9040}{44.2}\right)$$

$$= 41.5 \text{ ksi} < 56 \text{ ksi} \qquad \text{OK}$$

STEP 7: Check haunch web and beam web shear capacities.

Use Equation 6.12 to check the haunch web width-thickness ratio:

$$\frac{a\sin\theta}{t_{hw}} = \frac{18 \sin 31°}{0.48} = 19.3 \leq \frac{260}{\sqrt{F_y}} = 36.8 \qquad \text{OK}$$

The average shear stress in the haunch web can be computed using Equation 6.13

$$\tau_{hw} = \frac{aV_{pd}}{2(1+\nu)I_b}\left(\frac{L'}{2} - \frac{\beta}{\tan\theta}\left(\frac{d}{2}\right) + \frac{(1-\beta)a}{3}\right)$$

$$= \frac{18 \times 203.5}{2 \times (1+0.3) \times 9040}\left(\frac{305.3}{2} - \frac{0.93}{0.6} \times \frac{35.85}{2} + \frac{(1-0.93)\times 18}{3}\right)$$

$$= 19.5 \text{ ksi} < \phi(0.6F_y) = 27 \text{ ksi} \qquad \text{OK}$$

Use Equation 6.15 to compute the shear in the beam web as

$$V_{bw} = (1-\beta)V_{pd} = (1 - 0.93) \times 203.5 = 14.2 \text{ kips}$$

The above computation indicates that the welded haunch is very effective in reducing the beam shear at the column face. Nine existing high strength bolts (1-in. diameter A325 bolts) provide a shear strength of 120.6 kips.

STEP 8: Design Welds and Stiffeners.

Complete penetration groove weld (E71T-8 electrode with a specified CVN value of 20 ft-lb at -20°F) at both ends of the haunch flange are specified to transmit the haunch flange force.

Design the haunch web fillet weld:

$$v_{hw} = \tau_{hw}t_{hw} = 19.5 \times 0.48 = 9.4 \text{ kips/in.}$$

The required fillet weld size is

$$a_w \geq \frac{v_{hw}}{\phi(0.707)(0.60F_{yw})2} = \frac{9.4}{0.75(0.707)(0.6\times 70)2}$$

$$= 0.21 \text{ in.}$$

A 5/16-in. fillet weld size is sufficient.

Without beam web vertical stiffeners, the maximum concentrated compressive strength is governed by local web yielding:

$$\phi R_n = 1.0(2.5k + N)F_{yw}t_w$$

$$= 1.0(2.5 \times 1.875 + 0.77)(50)(0.625)$$

$$= 170.5 \text{ kips} < \beta V_{pd} = 189.3 \text{ kips} \qquad \text{NG}$$

Try a pair of 1/2-in. × 5¼-in. plates (A572 Gr. 50 steel) for the stiffeners. Check the width-thickness ratio:

$$\frac{b}{t} = \frac{5.25}{0.5} = 10.5 \leq \frac{95}{\sqrt{F_y}} = 13.4 \qquad \text{OK}$$

Treat the stiffened web as an axially compressed member with an effective length of $0.75h$ (h = 32.5 in.), a cross section composed of two stiffeners and a strip of the beam web having a width of $12t_w$ (see Figure 6.9).

$$A_{eff} = 2(5.25)(0.5) + 12(0.625)(0.625) = 9.94 \text{ in.}^2$$

$$I_{eff} = 0.5(5.25\times 2 + 0.625)^3/12 = 57.4 \text{ in.}^4$$

$$r = \sqrt{\frac{I_{eff}}{A_{eff}}} = \sqrt{\frac{57.4}{9.94}} = 2.40 \text{ in.}$$

$$\frac{KL}{r} = \frac{0.75h}{r} = \frac{0.75(32.5)}{2.40} = 10.2$$

$$\phi_c F_{cr} = 42.2 \text{ ksi}$$

$$\phi_c P_n = \phi_c F_{cr}(A_{eff}) = 42.2(9.94)$$

$$= 419 \text{ kips} > \beta V_{pd} = 189.3 \text{ kips} \qquad \text{OK}$$

Use complete joint penetration groove weld to connect each stiffener to the beam flange. Use two-sided 1/4-in. fillet welds to connect the stiffeners to the beam web.

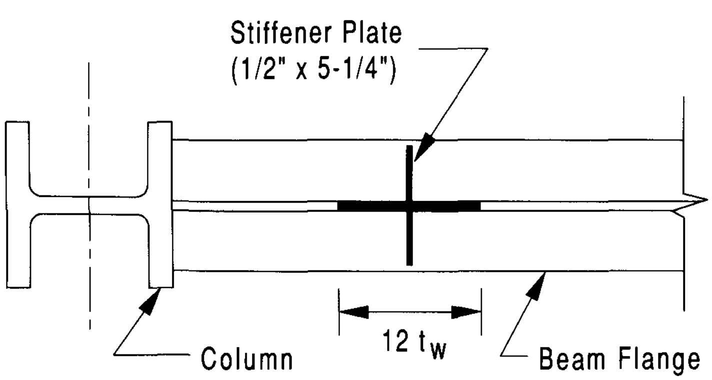


Figure 6.9 Stiffened Beam Web

Chapter 7
DESIGN OF BOLTED BRACKET MODIFICATION

When considering bolted bracket connections, several options are available regarding the type and location of the bracket. The options for the type of bracket are the haunch, pipe, or angle bracket. The options for the location include the bottom flange only, or both top and bottom flanges.

In previous tests, connections having a bracket attached to the bottom flange only and a high notch toughness full penetration groove weld at the top flange performed well (Kasai et al. 1997, 1998). However, in the NIST/AISC tests a similar connection, only using the low notch toughness E70T-4 electrode for the top flange weld, performed poorly. It appears that a bottom bracket only, in combination with replacement of the top flange weld with high notch toughness material, would be a viable solution (i.e., similar to the RBS in Chapter 5). However, the modification to be discussed in this chapter assumes no use of heat, as commented earlier in Section 2.3. For this reason, it is recommended that brackets be attached to both top and bottom flanges without modifying the pre-Northridge weld.

Use of a haunch bracket, pipe bracket, or strong double angle bracket at both the top and bottom flanges (see Figure 7. la to c) permits the development of plastic rotations

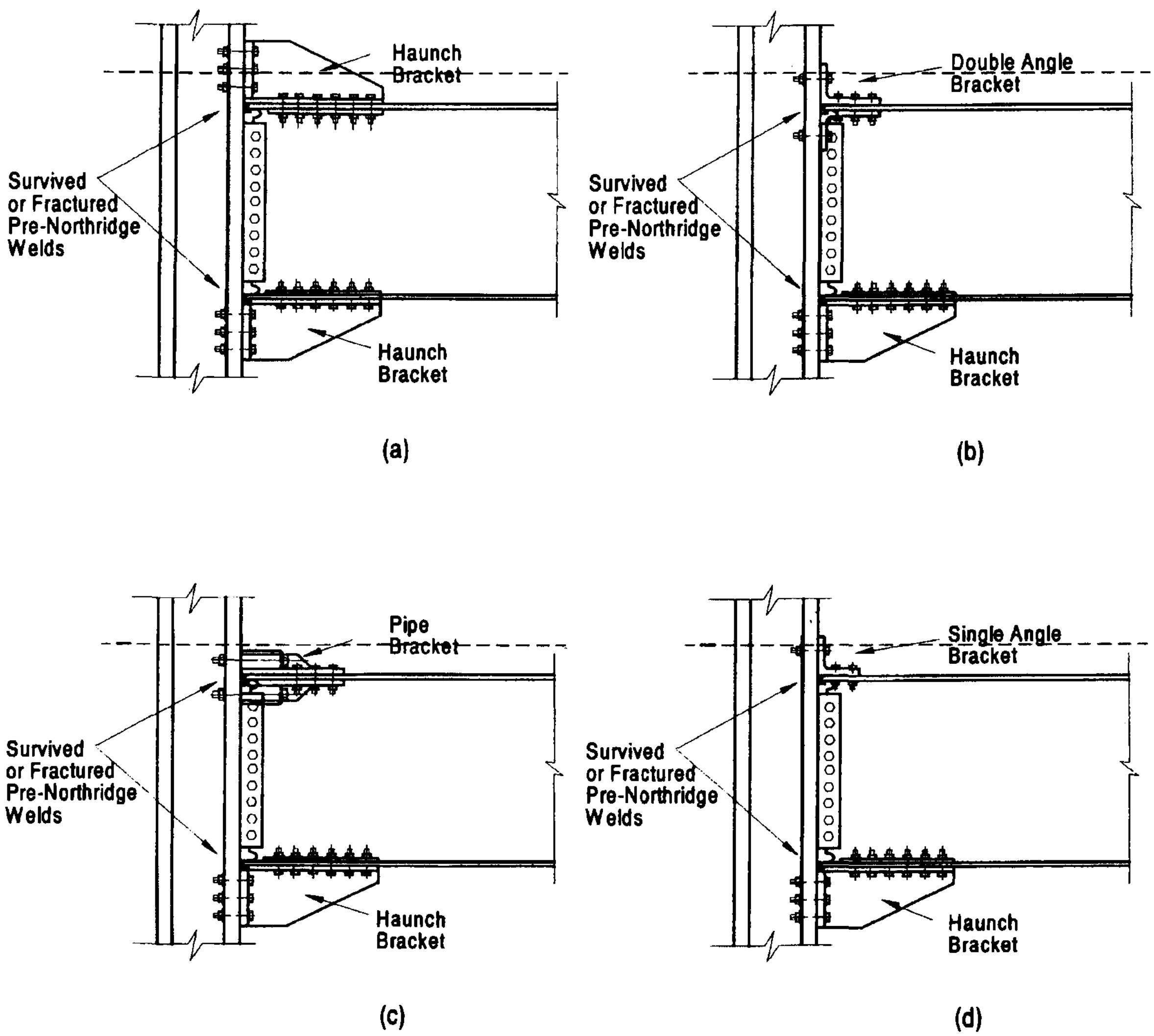


Figure 7.1 Possible Options for the Bolted Bracket Modification

on the order of 0.05 radian to 0.06 radian. The pipe bracket and angle bracket for the top flange do not require extensive removal of the concrete slab and may be concealed below the slab surface. However, in this Design Guide, we will consider only the strong double angle bracket for the top flange (Figure 7.lb). Compared with the pipe bracket, the double angle bracket requires minimum fabrication. For the bottom flange, a haunch bracket is recommended. Compared to either the pipe or double angle bracket, it has the beneficial effect of limiting stress and strain demands on the top flange (Section 7.1.6).

The combination of the top double angle bracket and bottom haunch bracket is considered to be the minimum modification to achieve an acceptable level of strength and ductility. The brackets are designed to be strong enough to resist the ultimate moment transferred from the fully yielded beam even after fracture of the pre-Northridge weld, and to assure beam plastic rotations of 0.05 radian to 0.06 radian as evidenced by such specimens. Moreover, since the bolt holes created in the beam flange cause early yielding of the flange net area and a part of the flange tensile force is also transmitted to the bracket, the present modification tends to limit the stresses at the flange weld, thereby preventing premature weld failure.

7.1 Minimum Recommended Bracket Design Provisions

The minimum modification shown in Figures 7.lb and 7.2 consists of a haunch bracket attached to the bottom flange and a double angle attached to the top flange. The CJP groove welds at both the top and bottom flanges are assumed to be of the pre-Northridge type, and their modification is not required.

As seen from the NIST/AISC test specimens LU-5 and LU-6 (Table 3.6), the modification successfully prevented failure of the CJP groove welds. However, due to the limited test data, unreliable performance of the pre-Northridge weld, and possible presence of unrecognizably fine weld cracks, one should consider the possibility of failure in these welds. Based on these observations, the modification to be discussed herein is intended to provide significant reserve connection strength even after weld failure.

The design assumes that either top or bottom flange fractured during the Northridge event, or it tends to fracture even after the retrofit due to the reasons stated above. Thus, the flange tension forces are assumed to be taken entirely by the attached bracket. Note that a sudden flange weld failure could produce a significant impact load on the brackets and bolts. However, as discussed earlier (Section 3.2.3), a full-size test which created this situation showed no detrimental effect of the impact on the bracket and bolts. Although more study is needed to confirm this point, it is felt that the brackets designed to be strong (and stiff) enough to take the entire portion of the moment transferred from the beam could also be effective in preventing fracture of the pre-Northridge flange weld and that they could sustain the impact load even if fracture occurs. The following sections describe the minimum recommended design provisions for the bolted bracket.

7.1.1 Preliminary Proportioning of Bolted Haunch Bracket

The overall configuration of the bolted haunch bracket is shown in Figure 7.2. The horizontal length a and vertical length b are determined from

$$a \cong (0.6 \text{ to } 0.7)d \tag{7.1}$$

$$b \cong (0.45 \text{ to } 0.55)d \tag{7.2}$$

where

d = the beam depth.

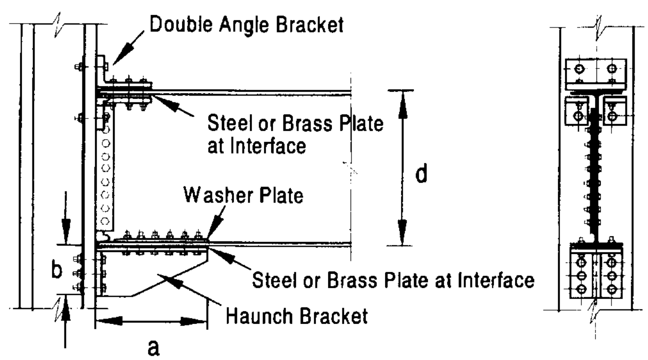


Figure 7.2 Minimum Recommended Modifications and Overall Dimensions

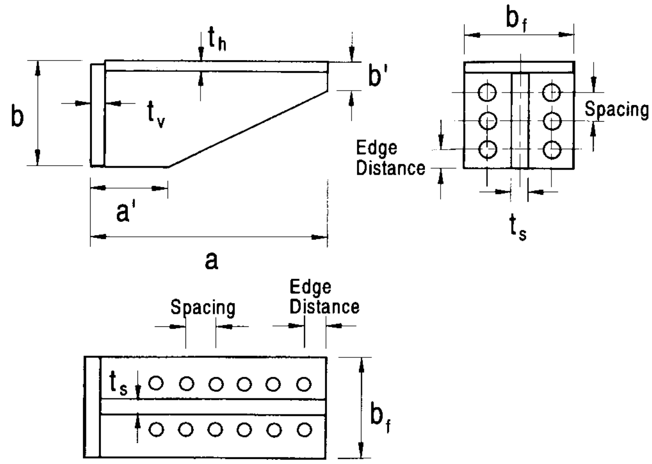


Figure 7.3 Dimensions of Haunch Bracket

The haunch stiffener taper is determined by the dimensions a' and b' (see Figure 7.3) as given by

$$a' \geq 3t_v \tag{7.3}$$

$$b' \geq 2.5t_h \tag{7.4}$$

where a' and b' are the lengths of the horizontal and vertical cuts of the haunch as shown in Figure 7.3, and

t_v = thickness of the vertical leg

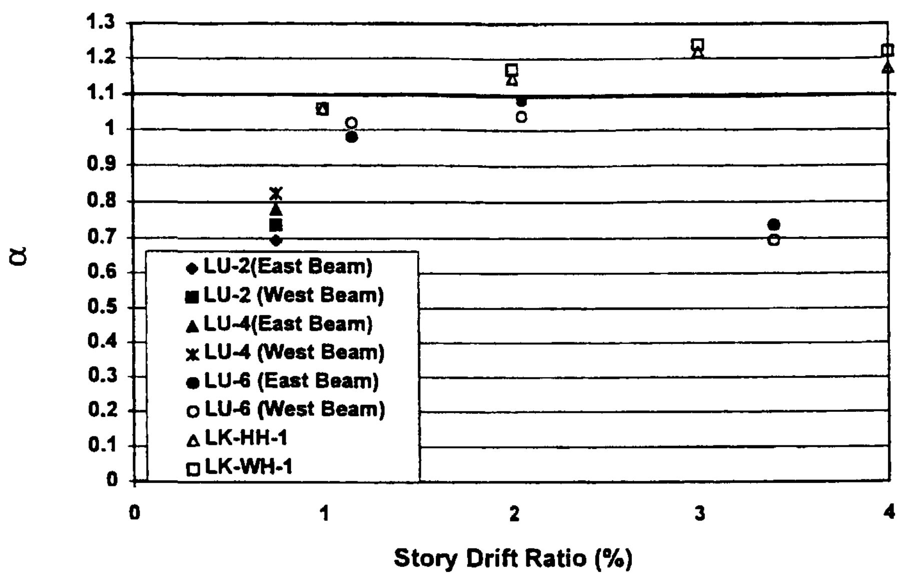
t_h = thickness of the horizontal leg.

The thickness of the horizontal leg, vertical leg, and haunch stiffener plate, t_h, t_v, and t_s, respectively, are given as:

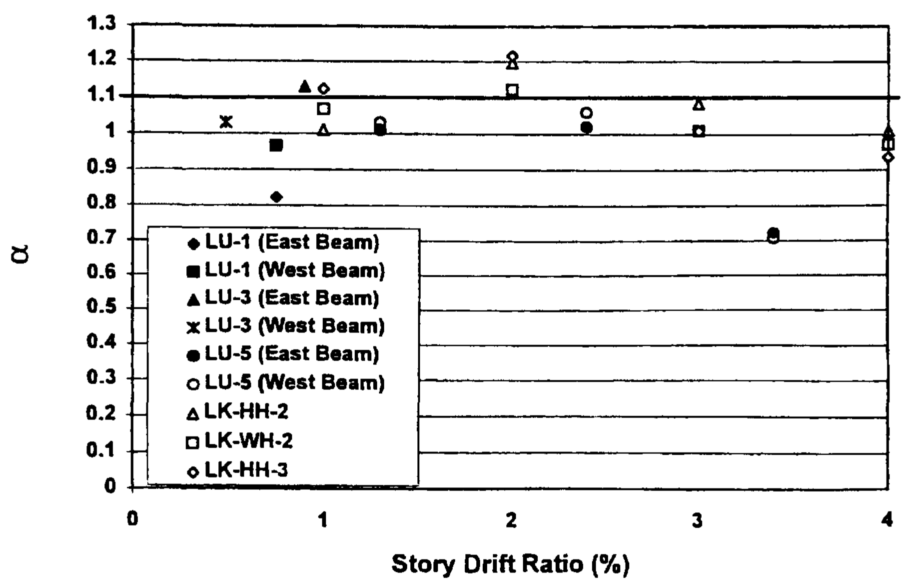
$$t_h \geq t_f \tag{7.5}$$

$$t_v \geq 1.5t_h \tag{7.6}$$

$$t_s \geq 1.5t_h \tag{7.7}$$



(a) Small Beam (LU: W30x99 Beam, LK: W16x40 Beam)



(b) Large Beam (W36x150 Beam)

Figure 7.4 α versus Story Drift Ratio for the Bolted Bracket

where

t_f = beam flange thickness.

The configuration satisfying Eqs. 7.3 to 7.7 is likely to pass the later strength check for the haunch stiffener as well as vertical leg (Section 7.1.5). These equations reflect dependency of the bracket design on the beam strength. Both experiments and analyses indicated reasonableness of the equations, assuming that the haunch bracket is made from ASTM Grade 50 steel and the beam from A36 steel having the yield stress similar to that explained in Section 4.1.

7.7.2 *Beam Ultimate Forces*

Further proportioning of the haunch bracket is based on the design moment, M_{pd}, and shear, V_{pd}, that develop at the critical plastic section which is taken at the tip of the haunch bracket (see Section 4.2). M_{pd} is given by Eq. 4.2 in which the factor a is intended to account for strain hardening (see Section 4.3.1). Plots of a versus Story Drift Ratio for subassemblage tests using the W16X40, W30X99, and W36x 150 beams are shown in Figure 7.4. The limited experimental study of the bolted bracket suggests $\alpha = 1.1$ as a reasonable estimate. Thus, the design moment at the critical plastic section (i.e., at the tip of the haunch bracket) may be expressed as

$$M_{pd} = 1.1 Z_b F_{ye} \qquad (7.8)$$

where

Z_b = plastic section modulus of the beam, and
F_{ye} = expected yield stress of the beam flanges as determined in Section 4.1.

The shear at the critical section, V_{pd}, is obtained from Eq.4.3.

7.1.3 *Haunch Bracket Forces at Beam Interface*

Figure 7.5 shows the free-body diagram of the beam and haunch bracket due to the positive and negative design moment at the critical plastic section, M_{pd}. The haunch bracket generates forces at the beam interface (i.e., between beam and bracket) as well as column interface (i.e., between column and bracket). The angle bracket attached to the top flange is assumed to generate a horizontal force only.

Horizontal Force. When M_{pd} is positive (Figure 7.5a), the bottom flange tension force is resisted entirely by the bracket due to the conservative assumption of bottom flange weld failure. If the resistance offered by the shear tab is ignored, the tension force acting on the bracket, $H_{hb}{}^{+}$, is conservatively estimated to be

$$H_{hb}{}^{+} \approx M_{pd}/d \qquad (7.9)$$

When M_{pd} is negative (Figure 7.5b), the flange compression force is partially resisted by the column face against which the flange bears. It is difficult to estimate the bearing force since it depends on the magnitude of the initial opening between the flange end and column face (caused by the possible weld failure), as well as relative horizontal movement between the beam flange and bracket. Based on widely scattered test data of the bearing force, it is conservatively estimated that the bracket resist at least 90% of the flange compression force. Accordingly, the compression force on the bracket, $H_{hb}{}^{-}$, is

$$H_{hb}{}^{-} \approx 0.9 M_{pd}/d \qquad (7.10)$$

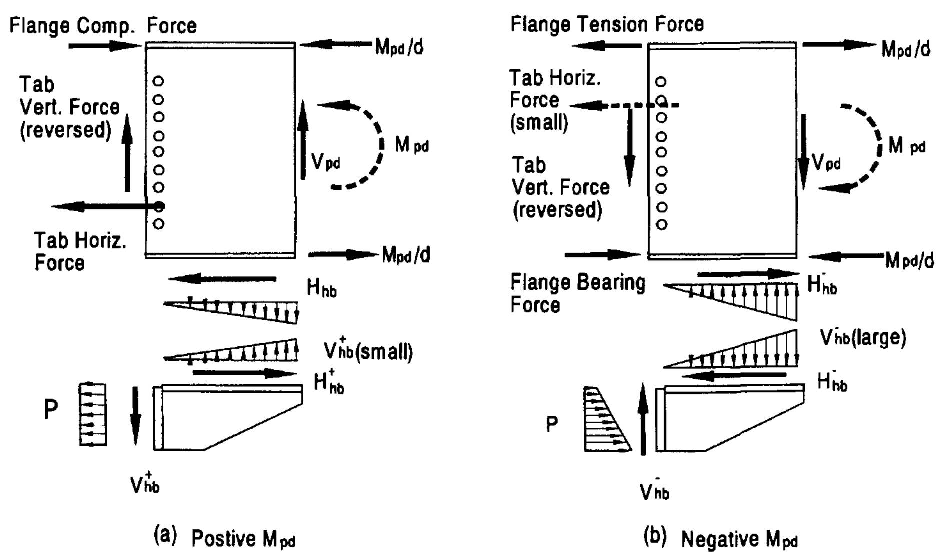


Figure 7.5 Free-Body Diagram ofBeam and Bracket

Vertical Force. The bracket is also subjected to a vertical force and localized moment at the beam interface (Figure 7.5). It is found from experiment and analysis that these forces are roughly proportional to V_{pd}.

When M_{pd} is positive (Figure 7.5a), the bracket is under tension force $H_{hb}{}^{+}$, thus the bracket contact force at the column interface is significantly reduced. This causes a reduction in friction force at the column interface. Further, the beam upward movement caused by the upward beam shear is resisted by the bottom beam flange as evidenced by partial separation at the beam interface observed both experimentally and analytically. These factors suggest limited upward force transfer from the beam to the bracket.

In contrast, when M_{pd} is negative (Figure 7.5b), the friction resistance at the column interface increases significantly. Further, the beam downward movement is resisted by the stiff haunch through contact at the beam interface. These factors suggest significant downward force transfer to the bracket. Both experiment and analysis indicate that this case is more critical for the haunch bracket than when M_{pd} is positive. As mentioned, the downward shear, $V_{hb}{}^{-}$, is found to be roughly proportional to the beam shear, or

$$V_{hb}{}^{-} \approx \beta V_{pd} \qquad (7.11)$$

Extensive data collection from many rosette strain gages on the haunch as well as three-dimensional nonlinear finite element analyses were performed to estimate the vertical force at the beam interface. It was found that a reasonable upper bound estimate for β is 1.7 when the haunch bracket is attached to the bottom flange of the beam only. Also, one could use the lower β of 1.4 if haunch brackets are attached at both top and bottom flanges, since the two brackets share the beam vertical force.

Moment. At the beam interface, local moments such as shown in Figures 7.5a and 7.5b develop due to the positive and negative beam moments, respectively. It was found that the moments are roughly proportional to the vertical force therein. Thus, an idealized triangular distribution of the vertical force obtained above is suggested to represent the effect of the moment (see Figure 7.5).

7.1.4 Preliminary Design of Haunch Bracket Bolts

Bolt Forces. The bolts used to attach the bracket to the beam will be referred to as "beam bolts" and those that attach the bracket to the column as "column bolts." The sizes and locations of these bolts are determined by using the interface horizontal force, H_{hb}, discussed above. The total shearing force of the beam bolts, $P_{b,bolt}$, is larger under the positive beam moment (compare Eqs. 7.9 and 7.10) and, therefore, $P_{b,bolt}$ is given by

$$P_{b,bolt} = H_{hb}{}^{+} \qquad (7.12)$$

Assuming all beam bolts share equally, each beam bolt shear force is equal to $P_{b,bolt}$ divided by the number of beam bolts.

The total tension force of the column bolts, $P_{c,bolt}$, is related to $H_{hb}{}^{+}$. Prying action at the column interface must be considered. Both experiment and analysis (Kasai et al. 1997) indicate that the prying force is at most about 30% of the applied force when the column bolts are spaced as described herein. Thus, a preliminary estimation for the bolt tension force is:

$$P_{c,bolt} \approx 1.3H_{hb}{}^{+} \qquad (7.13)$$

Both experiment and three-dimensional analysis show that the two column bolts located nearest the horizontal leg of the bracket develop somewhat larger tension forces than the other bolts. This is because these two bolts are subjected to the tension force transmitted by not only the haunch stiffener but also by the horizontal leg (Figure 7.3). However, analysis indicates considerably lower tension stress in the horizontal leg as compared to the haunch stiffener, which may account for the small difference among the bolt forces. This may be explained by noting in Figure 7.5a that the horizontal and vertical forces at the beam interface produce moments at the column interface which tend to cancel.

Due to these factors, the tension forces for the column bolts are assumed to be equal. Therefore, each bolt tension force may be computed as $P_{c,bolt}$ divided by the number of column bolts. Section 7.1.5 provides a method to realize this assumption.

Bolt Sizes. AISC LRFD Tables 8-11 and 8-15 list design strengths ϕR_n for bolts in shear and in tension, respectively, where ϕ = 0.75. The use of such low values may not be necessary, since capacity design is considered in this chapter and the bolt forces are essentially limited by the beam strength which is believed to be estimated conservatively. Further, traditional design for bearing bolts in shear ignores the resistance provided by friction at the interface, and should therefore be conservative.

Considering these factors, it is proposed to use $\phi = 0.9$ for both the beam bolts and column bolts. The bearing strengths of the base metal should be checked for the beam bolt design. The tests show that the bearing design strength provided by AISC may be insufficient to prevent deformation of the bolt hole under severe cyclic load (Kasai et al. 1998). Thus, for the AISC LRFD Eq. J3-la, it is proposed to use the factor of 1.8 instead of 2.4 (by using ϕ = 0.9).

Bolt Locations. Bolt locations (Figure 7.3) are determined considering the AISC LRFD minimum spacing and edge distance requirements. Further, entering and tightening clearances given by AISC Table 8-4 (Volume 2,1994) should be considered. The column bolts should be located as close as possible to the column web in order to reduce the prying action as well as bending of the column flange. Also, they should be made as close as possible to the beam flange for an efficient load transfer. The location should

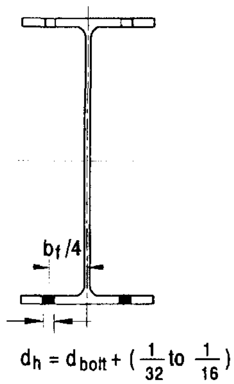
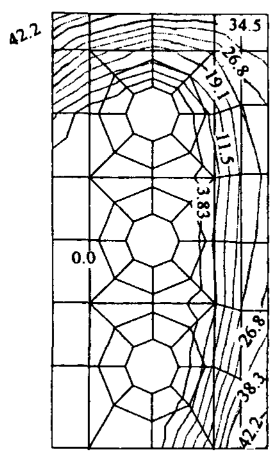
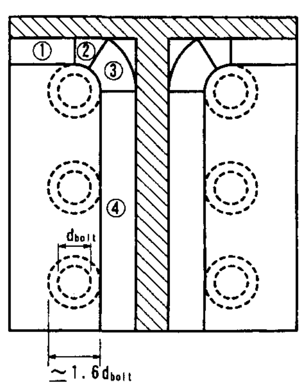


Figure 7.6 Net Area of Flanges



(a) Separation of Bracket from Column Face (x 0.0001 inches, Finite Element Analysis)



(b) Yield Lines for Limit Analysis

Figure 7.7 Deformation of Haunch Bracket Vertical Leg

also be checked for drilling clearances. The beam bolts should not be located too close to the beam flange edge to reduce the possibility of net area fracture. It is suggested that the bolt line be located at a distance of about $b_f/4$ from the center line of the beam flange (Figure 7.6).

7.1.5 Haunch Bracket and Bolts Check

The haunch bracket is subjected to combined axial force and bending moment as shown in Figure 7.5. The vertical leg and column bolts are critical under the tension force created by a positive moment (Figure 7.5a). The haunch stiffener is critical under the large bending moment and compression force created by a negative moment (Figure 7.5b). These components, preliminarily sized in Sections 7.1.1 and 7.1.4, are checked herein.

Positive Bending Case. The positive bending case produces a tension force which tends to separate the vertical leg from the column face. Figure 7.7a shows the contours of the separation, or out-of-plane deformation, of the vertical leg, which was analytically obtained via three-dimensional finite element analysis of a subassembly similar to the NIST/AISC specimen with W36 beam and bottom haunch bracket (Kasai et al. 1997). The result corresponds to the ultimate state of the beam, in which the vertical leg was almost elastic. Additionally, finite element analyses taken to the fully-plastic state of the leg indicated that the deformation pattern in Figure 7.7a remains similar. Considering these results, the plastic capacity of the leg can be calculated using yield lines and corresponding four yielded plate segments such as shown in Figure 7.7b.

However, a simpler and reasonably accurate approach is proposed which uses only two yielded plate segments as shown in Figure 7.8a. One yield line is defined along the toe of the supplemental fillet weld between the vertical leg and either the horizontal leg or haunch stiffener (Section 7.1.7). Another yield line is defined at the edges of the bolt head or nut whose width across flats is about 1.6 times the bolt diameter (Figures 7.7b and 7.8a). Each plate segment's yield capacity is obtained using plastic theory considering moment-shear interaction based on the von Mises yield criterion (Figure 7.8b).

At the fully plastic state the end moments of the plate are equal, thus, the end moment is equal to the shear times half the plate length (Figure 7.8b). By using this relationship, the moment-shear interaction equation can be expressed in terms of the shear force and the plate length only. Accordingly, the yield shear capacities, V'_{p1} and V'_{p2}, of the plate segments 1 and 2 are expressed as follows:

$$
\begin{aligned}
V'_{p1} &= \left(\sqrt{\alpha_1^2 + 1} - \alpha_1\right) V_{p1} \\
V'_{p2} &= \left(\sqrt{\alpha_2^2 + 1} - \alpha_2\right) V_{p2}
\end{aligned}
\tag{7.14}
$$

where

$$V_{p1} = 0.6 w_1 t_v F_y \qquad V_{p2} = 0.6 w_2 t_v F_y \qquad (7.15)$$

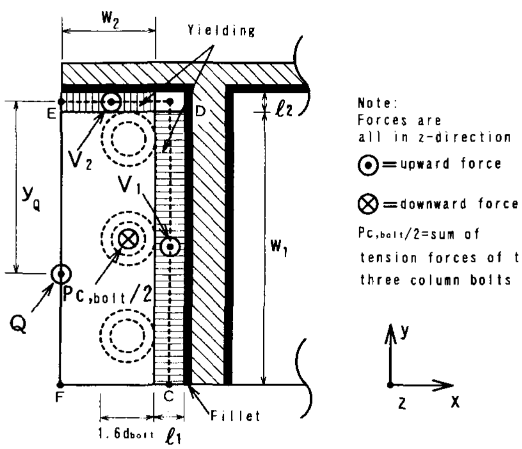
$$\alpha_1 = 0.6 l_1 / t_v \qquad \alpha_2 = 0.6 l_2 / t_v \qquad (7.16)$$

Note that $V'_{p1} \leq V_{p1}$ and $V'_{p2} \leq V_{p2}$ where V_{p1} and V_{p2} are the yield shear capacities when the bending moment is zero, and

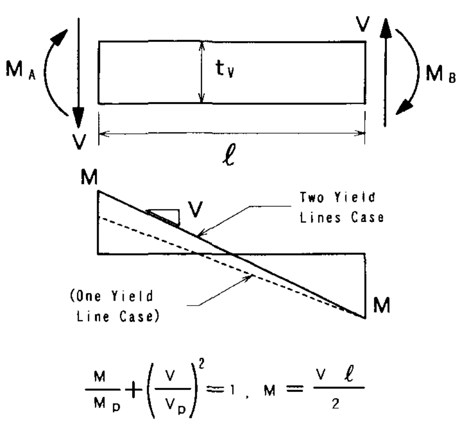
w_1 and w_2 = widths of plate segments 1 and 2,
l_1 and l_2 = lengths of plate segments 1 and 2, and
F_y = yield strength of the vertical stiffener steel.

It is required that the tension yield capacity of the vertical leg be greater than $H_{hb}{}^{+}$, i.e.,

$$\Omega_v = \frac{H_{hb}{}^{+}}{2(V'_{p1} + V'_{p2})} \leq 1 \qquad (7.17)$$



(a) Forces on Free Body C-D-E-F



(b) Moment and Shear Iteration

Figure 7.8 Simplified Analysis for Checking of Vertical Leg and Column Bolts

Ω_v is the load limiting factor of the vertical leg. When $\Omega_v < 1$, only one end of the plate segment will be fully yielded under the applied force, $H_{hb}{}^{+}$, and the moments at the ends will be unequal (Figure 7.8b). The end closer to the bolt typically develops a smaller moment. However, it can be shown that the prying force and bolt tension force under the applied force $H_{hb}{}^{+}$ can be conservatively estimated by assuming equal end moments (i.e., overestimating the moment of the end closer to the bolt). This assumption is adopted here since it simplifies the calculation. Consider shear forces V_1 and V_2 in plate segments 1 and 2, respectively.

$$V_1 = \Omega_v V'_{p1} \qquad V_2 = \Omega_v V'_{p2} \qquad (7.18)$$

The analysis proposed here is based on Igarashi et al.'s method (1985) to calculate the strength of bolted tube flanges. Using V_1 and V_2 determined from Eq. 7.18, consider three equilibrium equations for a free body C-D-E-F (Figure 7.8a).

$$V_1 + V_2 + Q - P_{c,bolt}/2 = 0 \qquad (7.19)$$

$$V_1 \times x_1 + V_2 \times x_2 - (P_{c,bolt}/2) \times x_3 = 0 \qquad (7.20)$$

$$V_1 \times y_1 + Q \times y_Q - (P_{c,bolt}/2) \times y_3 = 0 \qquad (7.21)$$

where

x_1, x_2, and x_3 = x-distances from line E-F to the loads V_1, V_2, and $P_{c,bolt}/2$.
y_1, y_Q, and y_3 = y-distances from line D-E to the loads V_1, Q, and $P_{c,bolt}/2$.
Q = prying force.

Equations 7.19 to 7.21 represent the force equilibrium in z-direction and moment equilibrium with respect to lines E-F and D-E, respectively, which give the solution for the three unknowns; bolt force, $P_{c,bolt}/2$, prying force, Q, and location of the prying force, y_Q. Based on experimentally and analytically observed typical deformation patterns of the vertical leg as well as bolts, it is reasonable to assume the prying force at the vertical edge E-F, similar to the consideration for a typical and simpler bolted connection involving prying.

The value of y_Q obtained should be small enough such that location of the force Q remains within line E-F. As a matter of fact, such a statically admissible solution reflects a small overturning moment of the vertical leg, leading to reasonably even forces of the three bolts shown in Figure 7.8. If the value of y_Q is such that the force Q is outside line E-F, it suggests that V_2 is too large compared with V_1 and/or w_2 is too large. These conditions tend to concentrate the bracket tension force on the bolt closest to the horizontal leg, and thus indicate the need to revise the vertical leg proportion. In this manner, the value of y_Q could reflect the appropriateness of vertical leg design.

Negative Bending Case. Longitudinal compressive stresses along the inclined edge of the haunch stiffener become large when the beam moment is negative (Figure 7.5b). This trend may be qualitatively explained using a truss-like model under the vertical force acting on the beam interface. However, the interface is subjected not only to vertical load, but also to moment and horizontal load (Figure 7.5b). To find the stresses in the haunch under the complex loading, the bracket is modeled as a tapered beam having a cross-sectional shape of a "tee" (Figure 7.9).

A strength check of the haunch can be performed by comparing the applied moment and axial force with the full plastic capacity of the haunch. The moment and axial force at any cross section is easily calculated using statics considering the vertical force, V_{hb}^-, and horizontal compressive force, H_{hb}^-, which are assumed to be triangularly and uniformly distributed, respectively (Figure 7.5b).

The simplest check for a given haunch configuration would be to obtain the moment capacity under the applied axial force, H_{hb}^-, and to compare it with the applied moment calculated from statics as mentioned above. In zone 1 (Figure 7.9), the critical section lies at the toe of the supplemental fillet weld. In zone 2, one must check several cross section locations.

An example of the rectangular yield stress distribution at the fully plastic state is shown in Figure 7.9b. This example shows the capacity check of zone 1. It is convenient to calculate the moment with respect to the top surface of the horizontal leg. The moment M_{top} due to the forces acting at the beam interface is

$$M_{top} = V_{hb}^- \times (a - t_v - 3/8) \times 2/3 \qquad (7.22)$$

The value 3/8 in Eq. 7.22 indicates the size (inches) of the supplemental fillet weld between the haunch stiffener and vertical leg. The available yield moment capacity, $M_{top,p}$, in the fully plastic case is

$$M_{top,p} = F_y[(A_{comp,1} + A_{comp,2}) \times d_{comp} - A_{tens} \times d_{tens}] \qquad (7.23)$$

where

$$A_{comp,1} = H_{hb}^- / F_y \qquad (7.24)$$

$$A_{comp,2} = A_{tens} = (A - A_{comp,1})/2 \qquad (7.25)$$

and

$A_{comp,1}$ and $A_{comp,2}$ = areas yielded in compression, due to H_{hb}^- and the moment

A_{tens} = area yielded in tension due to the moment

d_{comp} and d_{tens} = distances from top to centroids of the compressive and tensile areas

A and F_y = total cross-sectional area, and material yield strength of the haunch stiffener and horizontal leg

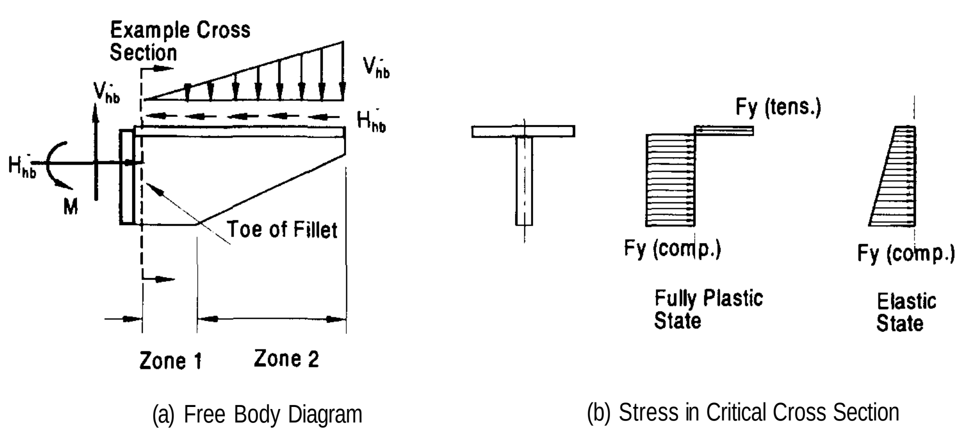
The design is satisfactory if

$$M_{top,p} \geq M_{top} \qquad (7.26)$$

A shear strength check should also be performed by comparing the applied shear force with the available shear strength of the cross section (e.g., Section 7.2).

7.7.6 *Angle Bracket Design*

Preliminary Proportioning of Angle Bracket. As explained earlier, the main purpose of the double angle bracket is to limit the top flange weld stress, and to provide large supplemental stiffness and strength in case of weld failure.



(a) Free Body Diagram

(b) Stress in Critical Cross Section

Figure 7.9 Calculation of Combined Moment and Axial Force Capacity of Haunch Stiffener

The use of more than one horizontal row of column bolts per angle is ineffective. On large columns with wide flanges, it is possible to place four bolts per row and this should be done, provided it does not reduce the column net area below the required amount.

It is preferred, however, to keep the number of column bolts as small as possible in order to reduce the time for drilling and to maximize the net area of the column section. Hence, the use of only two column bolts of relatively large size per row is considered herein (Figure 7.10). Also the number of beam bolts is limited by the size of the largest available rolled angle section (L8x8), and is at most four. However, in order to fasten large beams such as W30 and W36 sections, more than four beam bolts would be required. In such a case, an angle may be fabricated from a W section as will be illustrated in Section 7.2.

Angle Bracket Forces. Figure 7.10 shows the configuration of the reinforcement at the top flange. The tension force acting on the double angle bracket, H_{ab}^{+}, is

$$H_{ab}^{+} = H_{hb}{}^{+} \tag{7.27}$$

Equation 7.27 assumes that the top flange tensile force does not increase from the critical plastic section (at the tip of the haunch bracket) toward the column face. This is based on the free body diagram in Figure 7.5b, which indicates that the upward force $V_{hb}{}^{-}$ tends to reduce the top flange force at the column face. This could be also supported by the experimental observation that the bottom haunch bracket causes a long yield zone at the top flange extending up to the tip of the angle bracket, thereby controlling the strain demands and strain-hardening (specimens LU-5 and LU-6).

Application of AISC Design Method. The design of the double angle bracket follows the section titled "Hanger Connections" in Part 11 of AISC LRFD Manual, Volume II (1994). Therefore, the readers can refer to the manual for the details of the procedure. In summary, one should first assume the column bolt size to be 1.2 to 1.3 times $H_{ab}{}^{+}$ considering the effect of prying. To ensure adequate stiffness of the bracket, b_a in Figure 7.10 should be made

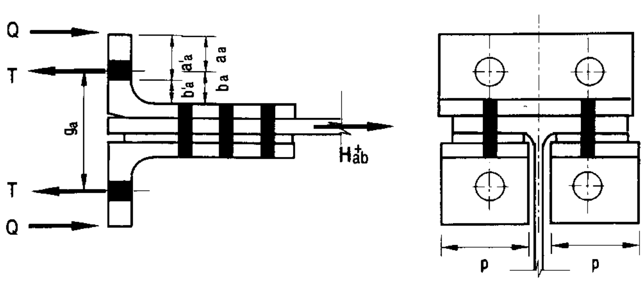


Figure 7.10 Dimensions of Angle Bracket

as small as the bolt entering and tightening clearances will permit. Then, assume a_a to be more than 1.25 b_a to assure the smallest prying force per AISC LRFD. Required vertical leg thickness, prying force Q and bolt tension force T in Figure 7.10 are calculated using the AISC formula.

Detailing Method. The column bolts and beam bolts are sized for T and H_{ab}^{+}, respectively. The number of beam bolts would be substantially less than that for the bottom haunch bracket because they are in double shear. Note, however, that the bearing stress at the flange hole increases to twice the magnitude as compared with the single shear case. The beam bolts and column bolts must be sized using the strength reduction factor ϕ = 0.9 as noted earlier.

When designing for a large beam section such as W36, the number of beam bolts required would be larger than can be accommodated by the largest rolled angle section available (leg length of 8 inches). In such a case it is recommended to cut a W-section and to fabricate the angle bracket as was done for specimens LU-5 and LU-6. The use of ASTM Grade 50 material (used for specimens LU-5 and LU-6) or higher is recommended in order to make the bracket compact. For the horizontal leg of the bracket, yielding of the gross section as well as the net section must be checked by following chapter B of AISC Manual Volume I (1994).

Note that the top flange weld often extends above the top surface of the flange. In such a case, grinding of the back (heel) of the angle may be required. Also, for the angle bracket on the bottom side of the beam top flange, a spacer plate of about 1/2 inch between the flange surface and bracket may be needed to accommodate the existing steel backing.

7.1.7 Requirements for Bolt Hole and Weld Sizes

Bolt Hole Sizes. The hole size for the beam bolts should be made as small as practical in order to limit the amount of slip at the beam interface which can occur at a low load. For this reason, drilling is mandated to create the hole. In order to save labor costs at the site, the holes for the haunch and angle brackets should be made in the shop. The bracket can then be used as a template for on-site drilling of the holes in the beam and column flanges. The desired diameter for the beam bolt hole is 1/32 in. to 1/16 in. greater than the bolt diameter, (e.g., Figure 7.6), and 1/8 in. greater for the column bolt. The latter provides fit-up tolerance.

To prevent a possible premature fracture of the net area, the following criterion should be satisfied:

$$A_n F_u \geq A_g F_{yef} \tag{7.28}$$

where

A_g = gross area of a beam or column flange,
A_n = net area of a beam or column flange, and
F_u = ultimate tensile strength of the flange material.

Equation 7.28 is intended to assure yielding of the gross section rather than premature net area fracture. Although Eq. 7.28 does not include explicitly the force increase due to strain-hardening of the gross section, it should not be considered unconservative. This is because a portion of the web can aid the flange net area by providing additional fracture resistance, and a portion of the flange tensile stresses would be transmitted to the bracket by virtue of the friction at the beam or column interface. Note that in the beam flanges of the test specimens violating Eq. 7.28, fracture did not occur until severe flange buckling propagated into the net area. These observations appear to agree with recent experimental findings (Masuda et al. 1998)

Reduction of column moment and axial force capacities due to column bolt holes need not be considered when checking column-beam moment ratio (Section 4.3.3), as long as Eq. 7.28 is satisfied. The recent Japanese study indicated that 30% to 40% loss of flange area due to bolt holes showed only about a 10% reduction in the yield moment capacity (Masuda et al. 1998), which could support this provision. Based on the experimental results, the minimum spacing of column bolts may be as small as 2⅓ times the bolt diameter.

Weld Sizes. A weld metal with a specified Charpy V-Notch toughness of 20 ft-lb at -20°F should be used for the haunch bracket fabrication. A sample weld detail is shown in Figure 7.11. A double-V bevel groove weld should be used to connect the haunch stiffener to the vertical leg. A single-V bevel groove weld may be used to connect the horizontal leg to the vertical leg. The end of the vertical leg may be offset up to 3/8 in. away from the outer surface of the horizontal leg to accommodate the steel backing, provided that at least 60% of the horizontal leg cross section is groove-welded. Although the current

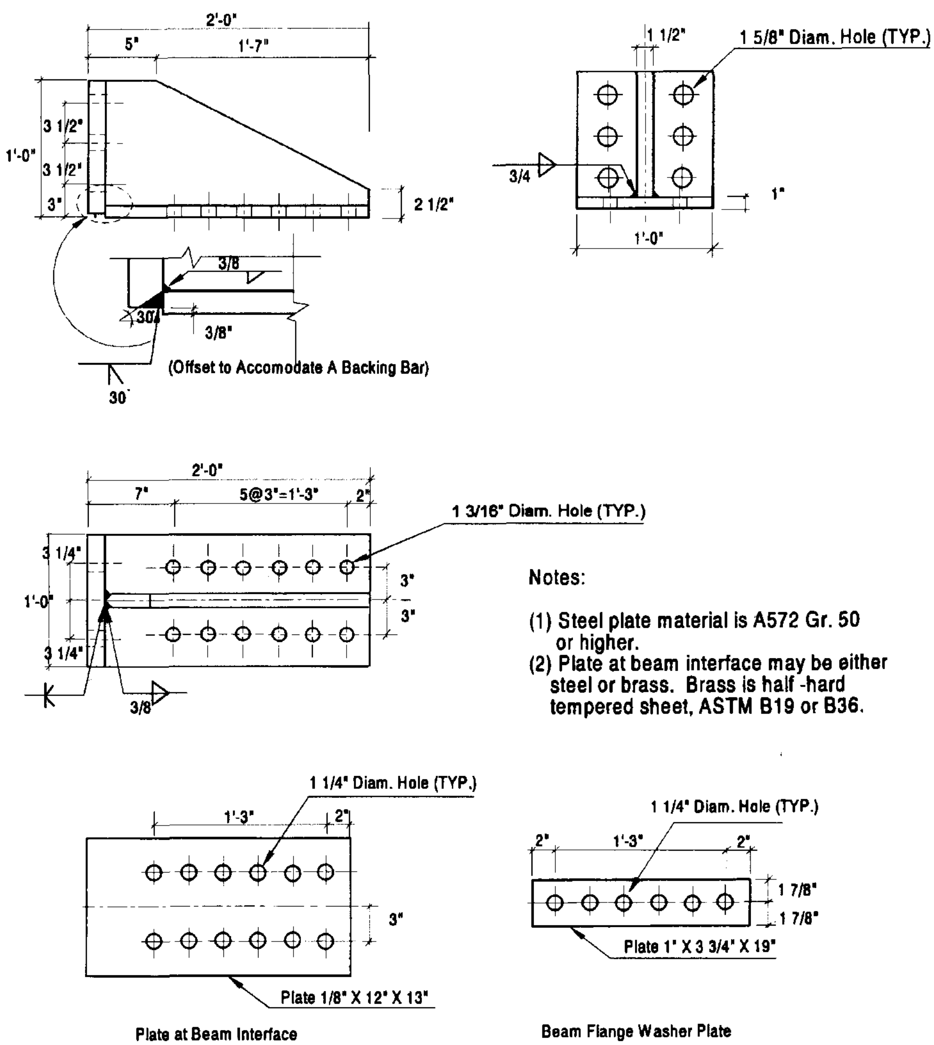


Figure 7.11 Haunch Bracket Detail

specification requires a 45° bevel except for submerged arc welding, a 30° bevel is preferred for the present detail only (Figure 7.11).

A minimum of 3/8 in. reinforcing fillet weld should be provided over the double-V and single-V welds (i.e., at the intersection lines formed by the inner surface of vertical leg and both the haunch stiffener and horizontal leg, see Figure 7.11). Fillet welds may be used to connect both sides of the haunch stiffener to the horizontal leg inner surface, and that size should be at least one-half the thickness of the haunch stiffener.

7.1.8 Column Panel Zone Check

The presence of bolted haunch brackets also creates an enlarged panel zone, and a check of panel zone shear strength may be unnecessary. The designer should use the recommendations of Section 7.5.2.6 of *Advisory No. 1* (FEMA 1997), by replacing the beam depth d_b term with $d_b + b$, where b is the vertical dimension of the haunch bracket.

7.1.9 Column Continuity Plate Check

Adding a pair of continuity plates at the beam flange level could reduce stress concentrations in the groove weld. Attachment of the bolted bracket without the addition of continuity plates could still reduce the weld stresses to the level at which weld fracture may be mitigated. If weld fracture occurs, the column flange will be subjected to bending because of the tension force applied by the column bolts. For this reason, a check of column flange bending strength should be performed according to local flange bending criteria given in Chapter 10, "Column Stiffening at FR and PR Moment Connections" of AISC manual (Volume II, 1994).

7.2 Design Example

Description of Existing Frame:

Beam:	W36×150	ASTM A36 steel
Column:	W14×426	ASTM A572, Gr. 50 steel

Centerline Dimensions:

- Story height: 12 ft
- Bay width: 30 ft

Existing Connection:

- Welded flange-bolted web connection to column flange—interior connection
- Beam flange groove welds: 70T-4 SS-FCAW with steel backing and weld tabs left in place
- Beam web connection:
  - 9-1 in. diam. A325 bolts
  - 5/8 in. × 5 in. × 27 ½ in. shear tab, connected to column with 5/16 in. fillet welds
- No supplemental web welds between shear tab and beam web
- No continuity plates, and no doubler plates

Building Constructed in Early 1980's

Section Properties:

W36×150: d = 35.85 in.
b_f = 11.975 in.
t_f = 0.94 in.
t_w = 0.625 in.
Z = 581 in.^{3}

W14×426: d = 18.67 in.
b_f = 16.695 in.
t_f = 3.035 in.
t_w = 1.875 in.
Z = 869 in.^{3}

Expected Yield Stress of Beam Flange:

W36×150 beam was specified as A36. No testing was conducted on steel samples from the building and no CMTRs are available. Therefore, estimate F_{ye} based on Equation 4.1 and Table 4.1. Thus:

$$F_{ye} = R_y F_y = 1.3 \times 36 = 46.8 \text{ ksi}$$

Materials Used for Retrofit:

Haunch Bracket:	ASTM A572, Gr.50 Steel. F_y 50 ksi, F_u = 65 ksi
Angle Bracket:	ASTM A572, Gr.50 Steel. F_y 50 ksi, F_u = 65 ksi Angle cut from W36X256 section, see Figure 7.12
Bolts:	ASTM A490 bolts of 1 ⅛ in., 1 ½ in., and 1 ¾ in. diameter Use ϕ = 0.9 in lieu of 0.75. Accordingly, scaling up the values in AISC ERFD Tables 8.11 and 8.15, design strengths are: ϕR_n = 67.2 kips for 1 ⅛ in. bolt (single shear, X-type) ϕR_n = 180 kips for 1 ½ in. bolt (tension) ϕR_n = 245 kips for 1 ¾ in. bolt (tension)

Connection Modification Design:

Provide haunch bracket at the bottom flange and double angle bracket at the top flange of the beam, respectively. Beam uniform dead load w = 0.6 kips/ft.

- Preliminary Proportioning of Haunch Bracket

$a = (0.6 \text{ to } 0.7)d$	Try $a = 24$ in.
$b = (0.45 \text{ to } 0.55)a$	Try $b = 12$ in.
$a' \geq 3t_v$, $b' \geq 2.5t_h$	Try $a' = 5$ in., $b' = 2.5$ in.
$t_h \geq t_f$, $t_v \geq 1.5t_h$, $t_s \geq 1.5t_h$	Try $t_h = 1.0$ in., $t_v = 1.5$ in., $t_s = 1.5$ in.

- Compute s_m and L':

$$s_m = a = 24 \text{ in.}$$

$$L' = 360 - 18.67 - 2 \times 24 = 293.3 \text{ in.}$$

- Beam Ultimate Forces

$$M_{pd} = 1.1Z_bF_{ye} = 1.1 \times 581 \times 46.8$$
$$= 29{,}910 \text{ kip-in.}$$

$$V_{pd} = 2M_{pd}/L' + wL'/2$$
$$= 2 \times 29{,}910/293.3 + 0.6/12 \times 293.3/2$$
$$= 211.3 \text{ kips}$$

- Haunch Bracket Forces at Beam Interface

$$H_{hb}^{+} = M_{pd}/d = 29{,}910/35.85 = 834 \text{ kips}$$

$$H_{hb}^{-} = 0.9M_{pd}/d = 0.9 \times 29{,}910/35.85 = 751 \text{ kips}$$

$$V_{hb}^{-} = \beta V_{pd} = 1.7 \times 211.3 = 359 \text{ kips}$$

- Preliminary Design of Haunch Bracket Bolts

Check against Positive Bending:

$P_{b,bolt} = H_{hb}^{+} = 834$ kips	Try 12 beam bolts, 1 ⅛ in. diam.
$P_{c,bolt} \approx 1.3H_{hb}^{+} = 1084$ kips	Try 6 column bolts, 1 ½ in. diam.

Flange bearing strength $12 \times \phi R_n = 12 \times \phi 1.8 d_{bolt} t F_u = 12 \times 0.9 \times 1.8 \times 1.125 \times 0.94 \times 65 = 1336 > 834$ kips OK

The drilled holes are 1/16 in. (or less) and 1/8 in. oversize for the beam and column bolts, respectively. Consider the minimum edge distance and spacing as well as entering and tightening clearance (Sections 7.1.4 and 7.1.7). Beam bolts are assumed to be tightened from inside the beam, and column bolts from outside the column. In this example, a direct tension indicator washer is provided at the head of the beam bolt, and larger clearance of 5.5 in. is used. See Figure 7.11 for the detail.

- Detailed Check for Haunch Bracket and Bolts

Positive Bending Case:

Consider 3/8 in. supplemental fillet weld: l_1 and l_2 = 0.925 and 0.425 in.; w_1 and w_2 = 10.2 and 3.95 in., and α_1 and α_2 = 0.370 and 0.170, respectively.

$$V'_{p1} = \left(\sqrt{\alpha_1^2 + 1} - \alpha_1\right)V_{p1}$$
$$= 0.696 \times (0.6 \times 10.2 \times 1.5 \times 50)$$
$$= 320 \text{ kips}$$

$$V'_{p2} = \left(\sqrt{\alpha_2^2 + 1} - \alpha_2\right)V_{p2}$$
$$= 0.844 \times (0.6 \times 3.95 \times 1.5 \times 50)$$
$$= 150 \text{ kips}$$

$$\Omega_v = \frac{H_{hb}^{+}}{2(V'_{p1} + V'_{p2})} = 0.887 < 1.0 \quad \text{OK}$$

$$\therefore V_1 = \Omega_v V'_{p1} = 284 \text{ kips, and}$$
$$V_2 = \Omega_v V'_{p2} = 133 \text{ kips}$$

Considering free body C-D-E-F in Fig. 7.8,

$$284 + 133 + Q - P_{c,bolt}/2 = 0$$

$$284 \times 4.41 + 133 \times 1.98 - (P_{c,bolt}/2) \times 2.75 = 0$$

$$284 \times 5.31 + Q \times y_Q - (P_{c,bolt}/2) \times 4.91 = 0$$

$$\therefore P_{c,bolt} = 1102 \text{ kips}, \quad Q = 134 \text{ kips}, \quad \text{and}$$
$$y_Q = 8.93 \text{ in.}$$

$P_{c,bolt}$ is only 2% over ϕR_n = 1080 kips (the solution is conservative, anyway). Location of Q is within line EF. OK

Negative Bending Case:

Consider first the tee cross section at toe of the fillet (Figure 7.9). The moment M_{top} at the top of the cross section is

$$M_{top} = V_{hb}^{-} \text{ X } (24 - 1.5 - 0.375) \text{ x } 2/3$$
$$= 5295 \text{ kip-in}$$

On the other hand, consider a fully plastic case under the compression H_{hb}^{-} = 751 kips.

$$A_{comp,1} = H_{hb}^{-}/F_y = 751/50 = 15.0 \text{ in.}^2$$

$$A_{comp,2} = A_{tens} = (A - A_{comp,1})/2$$
$$= (28.5 - 15.0)/2 = 6.75 \text{ in.}^2$$

The available yield moment capacity $M_{top,p}$ calculated at top is

$$M_{top,p} = F_y\{(A_{comp,1} + A_{comp,2}) \times d_{comp} - A_{tens} \times d_{tens}\}$$

$$= 50 \times (21.75 \times 5.12 - 6.75 \times 0.28)$$

$$= 5474 > M_{top} = 5295 \text{ kip-in.} \quad \text{OK}$$

Compare the available shear strength with the cross section (e.g., Section 7.2).

$$V_p = 0.6t_s bF_y = 0.6 \times 1.5 \times 12 \times 50$$

$$= 540 \text{ kips} > V_{hb}^- = 359 \text{ kips} \quad \text{OK}$$

Repeat the above for other cross sections by paying attention to the distributed H_{hb}^- and V_{hb}^- (Figure 7.9). The bracket size appears to be adequate.

- Preliminary Design of Angle Bracket
  Check against Positive Bending:

$P_{b,bolt} = H_{ab}^+ = H_{hb}^+ = 834$ kips — Try 8 beam bolts, $1\frac{1}{8}$ in. diam. (double shear)

$P_{c,bolt} \approx 1.3H_{ab}^+ = 1084$ kips — Try 4 column bolts, $1\frac{3}{4}$ in. diam.

Flange bearing strength (double shear) $8 \times \phi R_n = 8 \times \phi \times 1.8 d_{bolt} t F_u = 8 \times 0.9 \times 1.8 \times 1.125 \times 0.94 \times 65 = 890 > 834$ kips. OK

The drilled holes are 1/16 in and 1/8 in. oversize for the beam and column bolts, respectively. Considering minimum edge distance and spacing as well as entering and tightening clearance, create the double angle from a W36 × 256 section (three angles from a section, Figure 7.12): t_w =0.96 in., t_f = 1.73 in.

- Detailed Check for Angle Bracket and Bolts
  Check Vertical Leg and Column Bolt:
  Use the procedure in AISC LRFD Part 11 "Hanger Connections." The design is governed by p = 6 in. (the smallest of the three pieces of angle, Figure 7.12).

$$a_a = 3.125 \text{ in.} > 1.125 b_a$$

$$= 1.25 \times 2.415 = 3.02 \text{ in.} \quad \therefore \text{ Use } a_a = 3.02 \text{ in.}$$

$$a'_a = 3.02 + 1.75/2 = 3.90 \text{ in.,}$$

$$b'_a = 2.415 - 1.75/2 = 1.54 \text{ in.}$$

$$\rho = b'_a/a'_a = 0.395$$

$$R_{ut} = H_{ab}^+/4 = 834/4 = 209 \text{ kip/bolt}$$

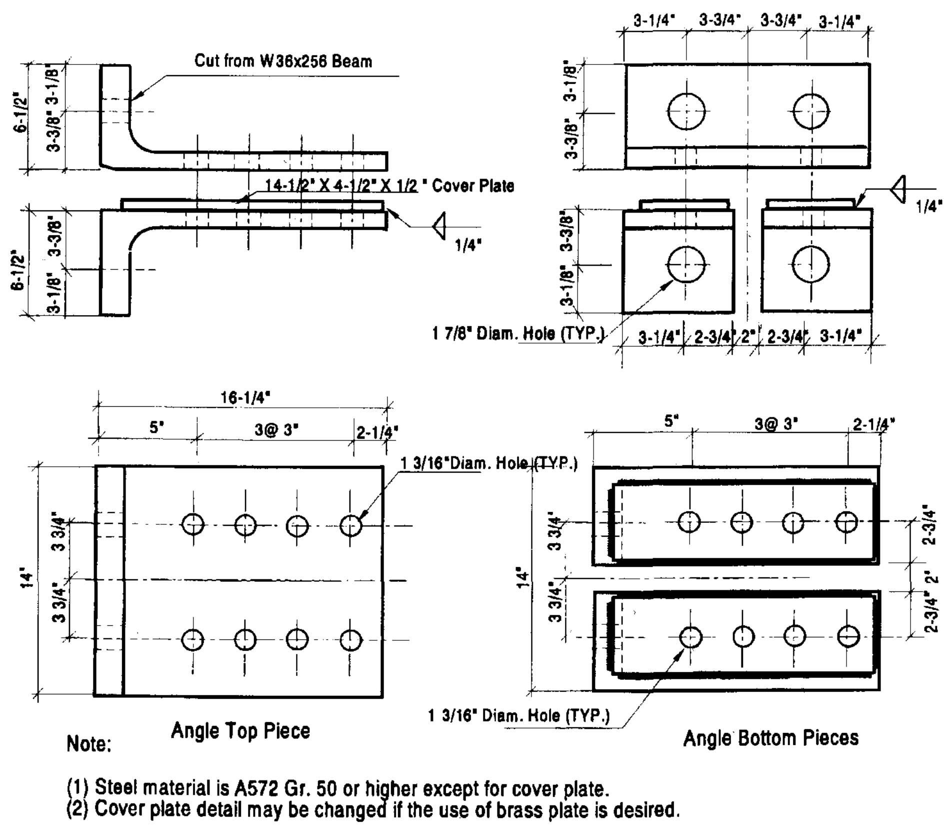


Figure 7.12 Angle Bracket Detail

$$\beta = \frac{1}{\rho}\left(\frac{\phi R_n}{R_{ut}} - 1\right) = \frac{1}{0.395}\left(\frac{245}{209} - 1\right) = 0.436 < 1.0$$

$$\delta = 1 - d'/p = 1 - (1.75 + 0.125)/6 = 0.688$$

$$\alpha' = \frac{1}{\delta}\left(\frac{\beta}{1-\beta}\right) = \frac{1}{0.688}\left(\frac{0.436}{1-0.436}\right)$$

$$= 1.12 > 1.0 \qquad \therefore \text{ Use } \alpha' = 1.0$$

$$\therefore\ t_{req} = \sqrt{\frac{4.44 R_{ut} b'_a}{pF_y(1+\delta\alpha')}} = \sqrt{\frac{4.44 \times 209 \times 1.54}{6 \times 50 \times (1 + 0.688)}}$$

$$= 1.68 < t_f = 1.73 \text{ in.} \quad \text{OK}$$

Since the thickness is satisfactory, the selected bolt is also adequate.

Check Horizontal Leg:

$$0.75F_uA_n = 0.75 \times 65 \times [6 - (1.125 + 0.0625)] \times 0.96$$
$$= 225 > 209 \text{ kips}$$

$$0.9F_yA_g = 0.9 \times 50 \times 6 \times 0.96 = 259 > 209 \text{ kips}$$

Thus, tension strengths of the plate gross and net areas are adequate (per Chapter B, AISC Specification)

- Beam and Column Flange Net Areas

Check Beam Flange:

$$F_uA_n = 65 \times 0.94 \times [12 - 2(1.125 + 0.0625)] = 588$$
$$> F_{yef}A_g = 46.8 \times 0.94 \times 12 = 528 \text{ kips}$$

Check Column Flange:

$$F_uA_n = 65 \times 3.035 \times [16.695 - 2(1.5 + 0.125)]$$
$$= 2856$$
$$> F_{yef}A_g = 50 \times 3.035 \times 16.695 = 2534 \text{ kips}$$

Thus, fracture of beam and column flange net area prevented (the web contribution is considered, Section 7.1.7).

- Column-Beam Moment Ratio (Section 4.3.3)

Assume no weld fracture, and calculate like welded haunch.

$$V_c = \frac{\sum[M_{pd} + V_{pd}(L - L')/2]}{h_b + d_p + h_t}$$

$$= \frac{2 \times [29{,}910 + 211.3 \times (360 - 293.2)/2]}{144}$$

$$= 513 \text{ kips}$$

$$M_c = M_{ct} + M_{cb} = V_c(h_t + h_b)$$

$$= V_c(144 - 35.85 - 12) = 49353 \text{ kip-in.}$$

No need to reduce column capacity per Section 7.1.7. Assuming $f_a = 10$ ksi,

$$\frac{\sum Z_c(F_{yc} - f_a)}{\sum M_c} = \frac{2 \times 869 \times (50 - 10)}{49353}$$

$$= 1.41 > 1.0 \quad \text{OK}$$

- Column Panel Zone

Panel zone strength should be adequate, since the presence of a haunch bracket creates an enlarged panel zone (Section 7.1.8).

- Continuity Plates

Existing connection was not provided with continuity plates. Use AISC LRFD Part 10, "Column Stiffening at FR and PR Moment Connections." The local flange bending strength ϕR_n is:

$$\phi R_n = \phi\left[\frac{b_s}{\alpha_m p_e}\right] t_f^2 F_y$$

$$= 0.9 \times \left[\frac{2.5 \times (2 \times 3.5)}{[1.36 \times (1.3125/1.5)^{1/4}] \times 1.3125}\right]$$

$$\times (3.035)^2 \times 50$$

$$= 4201 > H_{hb}{}^{+}$$

$$= 834 \text{ kips} \quad \text{OK}$$

Also, per Chapter K of the LRFD Specification, no continuity plates are required.

Chapter 8
CONSIDERATIONS FOR PRACTICAL IMPLEMENTATION

The selection of a particular connection modification scheme, the extent of work throughout the building lateral framing system, and its expected improvement to the seismic performance are critical decisions that are highly dependent upon the engineer's input on a rehabilitation project. The cost of the fabrication and erection work necessary to construct the structural details illustrated in this guide is usually only a small portion of the total cost of the rehabilitation work. Commonly, the costs associated with such related activities as the removal and restoration of collateral building finishes and services, tenant disruption, and the exhaust of welding fumes (when required) are the dominant cost factors. These considerations can also be more important for practical implementation than those associated only with structural fabrication and erection for an individual connection. Several issues are discussed here which relate to the costs associated with modifying a WSMF. Other factors, such as regional differences, seismic risk exposure, scheduling, and owner priorities, may impact the modification work.

As noted in Chapter 1, the issue of whether or not to rehabilitate a building is not covered in this document. If the decision is made to modify an exiting WSMF building, the question of whether to modify all, or only some, of the connections must also be addressed. This aspect is not covered here either as it is viewed as a decision which must be answered on a case-by-case basis. Further information and guidance on these issues may be obtained from *Interim Guidelines* (FEMA 1995) and *Advisory No. 1* (FEMA 1997).

8.1 Disruption or Relocation of Building Tenants

The disruption to the normal activities in a building may be a significant consideration when selecting a modification strategy. The following should be considered:

- Will the building be occupied while the connection modifications are being made?
- If the building is occupied, how much tenant disruption can be tolerated? Must tenants be relocated or will only spaces near the modifications need to be vacated?
- If access to the top beam flange is required, can the floors both above and below the work level be vacated?
- Must work be done during periods of occupancy or could it be done during off-hours?

8.2 Removal and Restoration of Collateral Building Finishes

Depending on the particular circumstances, the removal and restoration of collateral building finishes and services may be very costly. In general, these costs would be similar among the various modification strategies. However, the haunch and bolted bracket may pose interference problems which the RBS does not. Likewise, if access to the top flange of the beam is required, as in the case of the RBS and bolted bracket modifications, then additional costs would be likely. The following should be taken into consideration:

- Is the ceiling or soffit finish around the connection removable or hard framed?
- Are there any partition walls occurring near the modification which will be affected in order to gain access to the connection?
- Do any sprinkler lateral lines pass within working distance of the connection to be modified? If so, additional requirements may be imposed in order to shut down the sprinklers so modifications can be made.
- If the connections to be modified are located on the exterior of the structure, does the exterior finish allow access to the connection without removal or, if the exterior finish must be removed, can it be replaced without being noticeably different?
- Removal and replacement of spray-on fireproofing must be considered. In older buildings, care must be taken to prevent the crumbling of the spray-on fireproofing. The possible presence of asbestos also needs to be considered in older buildings.
- Do the mechanical ducts block access to the connection?
- Is there historical value to the structure—must the appearance be preserved?

8.3 Health and Safety of Workers and Tenants

When buildings remain occupied during rehabilitation, the safety and comfort of the occupants need to be considered. In all instances, the safety of the workers is an important consideration. Listed below are a few of the relevant safety issues:

- If the building tenants cannot be relocated and work must proceed during the night, will temporary modifications of the mechanical system have to be made in order to exhaust the fumes and bring in fresh air?

- If welding or cutting are required, is fire protection adequately addressed?
- What protection to the building contents needs to be considered?

8.4 Other Issues

There are a number of other issues which should also be considered in selecting modification strategy. Among them are:

- Is the construction more suited to the use of bolted or welded repairs?
- Is noise a factor?
- How will the new fittings be hoisted (by crane or elevator) and are there height or weight limitations?
- How will the dismantled pieces be removed?
- If partitions must be removed for access, are additional security requirements necessary?

REFERENCES

AISC (1997). *Seismic Provisions for Structural Steel Buildings,* American Institute of Steel Construction, Chicago, 1997.

"AISC Advisory Statement on Mechanical Properties Near the Fillet of Wide Flange Shapes and Interim Recommendations; January 10, 1997," *Modern Steel Construction*, Vol. 37, No. 2, February 1997, p. 18.

"AISC Initiates Research Into k Area Cracking," *Modern Steel Construction,* Vol. 37, No. 9, September 1997, pp. 23-24.

AISC (1994). *LRFD Manual of Steel Construction,* 2nd ed., Vol. I and II, AISC, Chicago, IL.

AISC (1993). *Load and Resistance Factor Design Specification for Structural Steel Buildings,* December 1, 1993, American Institute of Steel Construction, Chicago.

ATC (1988). *Guidelines for Cyclic Seismic Testing of Components of Steel Structures,* Report No. ATC-24, Applied Technology Council, Redwood City, California.

AWS (1996). *Structural Welding Code—Steel,* AWS D1.1-94, American Welding Society, Miami, FL, 1996.

Blodgett, O.W., Funderburk, R.S. and Miller, O.K. (1997). *Fabricator's and Erectors' Guide to Welded Steel Construction*, The James F. Lincoln Arc Welding Foundation, Cleveland, OH.

Blodgett, O.W (1998). "The Effects of Constraint on Ductility in Welded Beam-to-Column Connections," *Proceedings, 1998 National Steel Construction Conference,* American Institute of Steel Construction, Chicago.

Bondy, K.D. (1996). "A More Rational Approach to Capacity Design of Seismic Moment Frame Columns," *Earthquake Spectra,* Vol. 12, No. 3, August 1996, pp. 395-406.

Chen, S.J. and Yeh, C.H. (1994). "Enhancement of Ductility of Steel Beam-to-Column Connections for Seismic Resistance," Proceedings, SSRC 1994 Task Group Meeting and Technical Session, Structural Stability Research Council, Lehigh University, Bethlehem, Pennsylvania.

Deierlein, Gregory (1995). "Summary of Building Analysis Studies," Technical Report: *Analytical and Field Investigations of Buildings Affected by the Northridge Earthquake of January 17, 1994,* SAC 95-05, Part 1, December 1995.

Engelhardt, M.D., Winneberger, T, Zekany, A.J., and Potyraj, T.J. (1997). "Experimental investigation of dogbone moment connections," Proceedings of the 1997 National Steel Construction Conference, Chicago, 7-9 May 1997, American Institute of Steel Construction, Inc., Chicago, 1997.

FEMA (1995). *Interim Guidelines: Evaluation, Repair, Modification and Design of Steel Moment Frames,* FEMA-267, (SAC-95-02), Federal Emergency Management Agency, Washington, D.C., August 1995.

FEMA (1997a). *Interim Guidelines Advisory No. 1,* FEMA-267A (SAC-96-03), Federal Emergency Management Agency, Washington, D.C., March 1997.

FEMA (1997b). *Background Reports: Metallurgy, Fracture Mechanics, Welding, Moment Connections and Frame Systems Behavior,* FEMA-288 (SAC-95-09), Federal Emergency Management Agency, Washington, D.C., March 1997.

FEMA (1998). *Update on the Seismic Safety of Steel Buildings, A Guide for Policy Makers,* Federal Emergency Management Agency, Washington, D.C., 1998.

Frank, K.H. (1995). "The Physical and Metallurgical Properties of Structural Steels," *Background Reports: Metallurgy, Fracture Mechanics, Welding, Moment Connections and Frame Systems Behavior,* FEMA-288, (SAC-95-09), Federal Emergency Management Agency, Washington, D.C., March 1997.

Galambos, TV. and Ravindra, M.K. (1978). "Properties of Steel for Use in LRFD," /. *Structural Div., ASCE,* Vol. 194, No. ST9, September, 1978, pp. 1459-1469.

Goel, S.C., Stojadinovic, B. and Lee, K.-H. (1997). "Truss Analogy for Steel Moment Connections," *Engineering Journal*, Vol. 34, No. 2, AISC, pp. 42-53.

Grubbs, K.V. (1997). "The Effect of the Dogbone Connection on the Elastic Stiffness of Steel Moment Frames," Master's Thesis, Dept, of Civil Engineering, The University of Texas at Austin, August 1997.

ICBO (1988). *Uniform Building Code.* International Conference of Building Officials (ICBO), Whittier, CA, 1988

ICBO (1994). *Uniform Building Code.* International Conference of Building Officials (ICBO), Whittier, CA, 1994.

"ICBO Board Approves Emergency Structural Design Provisions" (1994). *Building Standards.* September-October, 1994, p. 26.

Igarashi, S., Wakiyama, K., Inoue, K., Matsumoto, T., and Murase, H. (1985). "Limit Design of High Strength Bolted Tube Flanges," *Journal of Structural and Construction Engineering (Transactions of AIJ),* Architectural Institute of Japan, No. 358, pp. 71-82, December (in Japanese).

Iwankiw, N.R., and Carter, C. (1996). "The Dogbone: A New Idea to Chew On," *Modern Steel Construction,* April 1996.

Kasai, K, Hodgon, I., and Mao, C. (1997). "Bolted Repair Methods for Fractured Welded Moment Connections," *Proceedings, Behavior of Steel Structures in Seismic Areas (Stessa '97),* pp. 939-946, Kyoto, Japan, 3-8 August.

Kasai, K., Hodgon, I., and Bleiman, D. (1998). "Rigid-Bolted Repair Methods for Damaged Moment Connections," *Engineering Structures,* Vol. 20, No. 4-6, pp. 521-532.

Kaufmann, E.J., Fisher, J.W., DiJulio, R.M., and Gross, J.L. (1997). "Failure Analysis of Welded Steel Moment Frames Damaged in the Northridge Earthquake," NISTIR 5944, National Institute of Standards and Technology, Gaithersburg, MD.

Kaufmann, E.J., Xue, M., Lu, L.W., and Fisher, J.W (1996) "Achieving ductile behavior of moment connections," *Modern Steel Construction,* American Institute of Steel Construction, Inc., January 1996.

Lee, C.H. and Uang, C.-M., "Analytical Modeling of Dual Panel Zone in Haunch Repaired Steel MRFs," *Journal of Structural Engineering,* Vol. 123, No. 1, pp. 20-29, ASCE, 1997.

Masuda, H., Tamaka, A., Hirabayashi, K., and Genda, I. (1998). "Experimental Study on the Effect of Partial Loss of Sectional Area on the Static Characteristics of H-Beams," *Journal of Structural and Construction Engineering (Transactions of AIJ),* Architectural Institute of Japan, No. 512, pp. 157-164, October (in Japanese).

Noel, S. and Uang, C.-M., "Cyclic Testing of Steel Moment Connections for the San Francisco Civic Center Complex," Report No. TR-96/07, Division of Structural Engineering, University of California, San Diego, 1996.

Paulay, T. (1997). "Discussion of a More Rational Approach to Capacity Design of Seismic Moment Frame Columns," *Earthquake Spectra,* Vol. 13, No. 2, May 1997. pp. 317-320.

Plumier, A. (1990). A New Idea for Safe Structures in Seismic Zones, *IABSE Symposium: Mixed Structures Including New Materials,* Brussels, pp. 431-436.

SAC (1995). "Analytical and Field Investigations of Buildings Affected by the Northridge Earthquake of January 17, 1994," *Technical Report SAC 95-04,* SAC Joint Venture, Sacramento, CA.

SAC (1996). "Experimental Investigations of Beam-Column Subassemblages," *Technical Report SAC-96-01, Parts 1 and 2,* SAC Joint Venture, Sacramento, CA.

SEAOC (1990). *Recommended Lateral Force Requirements and Commentary,* Structural Engineers Association of California (SEAOC), Sacramento, 1990.

Schneider, S. P., Roeder, C.W., and Carpenter, J.E. (1993). "Seismic Behavior of Moment-Resisting Steel Frames: Experimental Study," *Journal of Structural Engineering,* ASCE, Vol. 119, No. 6, June 1993, pp. 1885-1902.

Tremblay, R., Tchebotarev, N., and Filiatrault, A. (1997). "Seismic performance of RBS connections for steel moment resisting frames: influence of loading rate and floor slab," In: Proceedings; STESSA '97. Kyoto, Japan, 4-7 August 1997.

Uang, C.-M. and Bondad, D. (1996). "Dynamic Testing of pre-Northridge and Haunch Repaired Steel Moment Connections," Report No. SSRP 96/03, University of California, San Diego, La Jolla.

Yang, T. and Popov, E.P. (1995). "Behavior of pre-Northridge moment resisting steel connections," Report No. UCB/EERC - 95/08, Earthquake Engineering Research Center, University of California at Berkeley, 1995.

Youssef, N.F.G., Bonowitz, D., and Gross, J.L. (1995). "A Survey of Steel Moment-Resisting Frame Buildings Affected by the 1994 Northridge Earthquake," NISTIR 5625, National Institute of Standards and Technology, Gaithersburg, MD, April, 1995.

Yu, K., Noel, S., and Uang, C.-M., "Experimental Studies on Seismic Rehabilitation of pre-Northridge Steel Moment Connections: RBS and Haunch Approaches," Report No. SSRP-97/08, University of California, San Diego, La Jolla, 1997.

Zekioglu, A., Mozaffarian, H., and Uang, C.M., "Moment Frame Connection Development and Testing for the City of Hope National Medical Center," Proceedings, 1997 Structures Congress, ASCE, Portland, April 1997.

SYMBOLS

A_b = Area of beam section, in.^{2}

A_{hf} = Haunch flange area, in.^{2}

A_f = Gross area of beam flange, in.^{2}

A_n = Net area of beam flange, in.^{2}

F_u = Specified minimum tensile strength of steel, ksi

F_y = Specified minimum yield stress of steel, ksi

F_{ye} = Expected yield strength, ksi

$H_{ab}{}^+$ = Tension capacity of angle bracket, kips

$H_{ab,y}$ = Yield force of gross section of horizontal leg of angle bracket (no bending), kips

$H_{hb}{}^+$ = Tension force on bolted bracket, kips

$H_{hb}{}^-$ = Compression force on bolted bracket, kips

I_b = Moment of inertia of beam section, in.^{4}

L = Center-to-center spacing of columns, in.

L' = Beam span between plastic hinges, in.

L_{CL} = Distance between center of beam span and the centerline of the column, in.

M_{cb} = Column moment below connection, kip-in.

M_{ct} = Column moment above connection, kip-in.

M_F = Moment at the face of the column, kip-in.

M_p = Nominal plastic moment $(Z$ X $F_y)$, kip-in.

M_{pd} = Design plastic moment, kip-in.

P = Estimated maximum axial force in columns above and below connection due to combined gravity and lateral loads, kips

$P_{b,bolt}$ = Shear force on beam bolts (bolted bracket), kips

P_{bf} = Beam bottom flange axial force, kips

$P_{c,bolt}$ = Tension force on column bolts (bolted bracket), kips

P_{hf} = Haunch flange axial force, kips

R = Radius of reduced beam section cut, in.

R_y = Ratio of the expected yield strength, F_{ye} , to the specified minimum yield strength, 7_y

V_{bw} = Shear force in beam web, kips

V_c = Shear force in columns above and below connection, kips

V_G = Shear force due to gravity loads, kips

$V_{hb}{}^-$ = Downward shear force on bolted bracket, kips

V_{pd} = Design shear force, kips

Z_b = Plastic section modulus of beam section, in.^{3}

Z_c = Plastic section modulus of column section, in.^{3}

Z_{RBS} = Plastic section modulus of reduced beam section, in.^{3}

a = Length of welded haunch, length of bolted bracket, or distance from face of column to beginning of reduced beam section cut, in.

a' = Dimension used for bolted bracket stiffener, in.

a_a = Distance from corner of angle bracket to nearest edge of beam bolt head, in.

b = Depth of welded haunch, depth of bolted bracket, or length of reduced beam section cut, in.

b' = Dimension used for bolted bracket stiffener, in.

b_a = Distance from corner of angle bracket to nearest edge of column bolt head, in.

b_f = Flange width or width of bolted bracket, in.

b_{hf} = Haunch flange width, in.

c = Depth of reduced beam section cut, in.

d, d_b = Beam depth, in.

d_c = Column depth, in.

d_p = Depth of modified beam (i.e., includes haunch or bracket), in.

f_a = Allowable stress, ksi

h = Story height, in.

h_b = Distance from the bottom of the connection to the point of inflection in the column below the connection, in.

h_t = Distance from the top of the connection to the point of inflection in the column above the connection, in.

k = Haunch flange axial stiffness, kip/in.

k_a = Distance from the corner of angle bracket to the toe of the fillet, in.

s_c = Distance from face of column to critical plastic section, in.

t_a = Thickness of angle leg, in.

t_f = Flange thickness, in.

t_h = Thickness of horizontal leg of bolted bracket, in.

t_{hf} = Haunch flange thickness, in.

t_{hw} = Haunch web thickness, in.

t_s = Thickness of stiffener plate of bolted bracket, in.

t_v = Thickness of vertical leg of bolted bracket, in.

t_w = Web thickness, in.

w = Uniform beam load, plf

α = Strain hardening factor

β = Ratio of vertical component of haunch flange force to design shear force

$\beta_{min.}$ = Minimum ratio of vertical component of haunch flange force to design shear force

Δ_{CL} = Plastic deflection of beam or girder, in.

θ = Angle between haunch flange and beam flange, deg

θ_p = Plastic hinge rotation, rad

ϕ = Resistance factor

ABBREVIATIONS

AISC	American Institute of Steel Construction
ASTM	American Society for Testing Materials
AWS	American Welding Society
BFRL	Building and Fire Research Laboratory
CJP	Complete Joint Penetration
CMTR	Certified Mill Test Report
CVN	Charpy V-Notch
FCAW	Flux Cored Arc Welding
FEMA	Federal Emergency Management Agency
ICBO	International Conference of Building Officials
NEHRP	National Earthquake Hazard Reduction Program
NIST	National Institute of Standards and Technology
RBS	Reduced Beam Section
SAC	Joint Venture comprised of the Structural Engineers Association of California, the Applied Technology Council, and the California Universities for Research in Earthquake Engineering
SMAW	Submerged Metal Arc Welding
UBC	Uniform Building Code
WSMF	Welded Steel Moment Frame

Appendix A
DEFORMATION COMPATIBILITY CONSIDERATIONS FOR WELDED HAUNCH CONNECTION

A.1 Deformation Compatibility between Haunch Flange and Beam

Consider the beam free body shown in Fig. 6.4(a). The horizontal and vertical components of the beam deformation at the haunch tip (Point B) can be computed as follows. Define x as the distance of the beam section measuring from the haunch tip toward the column face. The beam bending moment in the haunch region [see Fig. 6.4(c)] is

$$M(x) = (L'/2 + x)V_{pd} - x(\beta V_{pd}) - (\beta V_{pd}/\tan\theta)\frac{d}{2} \tag{A.1}$$

This bending moment together with the axial force [$\beta V_{pd}/\tan\theta$ in Fig. 6.4(b)] produce a compressive stress in the beam bottom flange as follows:

$$f_b(x) = \frac{(L'/2 + x)V_{pd}}{I_b}\left(\frac{d}{2}\right) - \frac{x(\beta V_{pd})}{I_b}\left(\frac{d}{2}\right) - \frac{(\beta V_{pd}/\tan\theta)}{I_b}\left(\frac{d}{2}\right)^2 - \frac{\beta V_{pd}/\tan\theta}{A_b} \tag{A.2}$$

The horizontal component (u_B) of the beam deformation at the haunch tip is equal to the axial shortening of the beam bottom flange in the haunch region:

$$u_B = \int_0^a \frac{f_b(x)}{E}dx = \left(\frac{L'/2 - (\beta/\tan\theta)}{EI_b}\left(\frac{d}{2}\right)^2 a + \frac{(1-\beta)a^2}{2EI_b}\left(\frac{d}{2}\right) - \frac{\beta a/\tan\theta}{EA_b}\right)V_{pd} \tag{A.3}$$

Next, consider the vertical component of the beam deformation. Using the moment-area method, where the moment is expressed in Eq. (A.1), the vertical component is expressed as follows:

$$v_B = \int_0^a \frac{xM(x)}{EI_b}dx = \left(\frac{L'/2 - (\beta/\tan\theta)}{EI_b}\left(\frac{d}{2}\right)a^2 + \frac{(1-\beta)a^3}{3EI_b}\right)V_{pd} \tag{A.4}$$

Based on the haunch tip displacement, the shortening of haunch flange and its force can be determined. The axial shortening of the haunch flange can also be established by considering the haunch flange as a free body. Since the vertical component of the haunch flange force is βV_{pd}, the haunch flange axial force is equal to $\beta V_{pd}/\sin\theta$, and the resulting axial shortening should be expressed as follows:

$$\sqrt{(a-u_B)^2 + (b-v_B)^2} - l_{hf} = \frac{\beta V_{pd}/\sin\theta}{EA_{hf}}l_{hf} \tag{A.5}$$

where $l_{hf}(=\sqrt{a^2+b^2})$ is the haunch flange *undeformed* length. The left-hand side of the above equation can be simplified by ignoring the higher-order terms for the small deformation theory, and the resulting equation is:

$$u_B\cos\theta + v_B\sin\theta = \frac{\beta V_{pd}/\sin\theta}{EA_{hf}}l_{hf} \tag{A.6}$$

Solving the above equation for β yields the following expression:

$$\beta = \left(\frac{b}{a}\right)\frac{3L'd + 3ad + 3bL' + 4ab}{3d^2 + 6bd + 4b^2 + \dfrac{12I_b}{A_b} + \dfrac{12I_b}{A_{hf}\cos^3\theta}} \tag{A.7}$$

A.2 Deformation Compatibility between Haunch Web and Haunch Flange

Treating the triangular haunch web as a first-order finite element, the constant shear strain is

$$\gamma_{hw} = \frac{\partial u(x,y)}{\partial y} + \frac{\partial v(x,y)}{\partial x} \tag{A.8}$$

where the displacement fields $u(x, y)$ and $v(x, y)$ can be expressed as a function of the nodal displacements (u_B and v_B) at the haunch tip (point B in Figure 6.7):

$$u(x, y) = \left(1 - \frac{x}{a}\right)u_B \tag{A.9}$$

$$v(x, y) = \left(1 - \frac{x}{a}\right)v_B \tag{A.10}$$

Substituting Eqs. A.9 and A.10 into Eq. A.8 gives the following:

$$\gamma_{hw} = -\frac{v_B}{a} \tag{A.11}$$

Substituting Eq. A.4 for v_B and multiplying both sides of the above equation by the shear modulus [$= E/2(1 + \nu)$] gives the following shear stress in the haunch web:

$$\begin{aligned}\tau_{hw} &= \frac{E}{2(1+\nu)}\left(\frac{L'/2 - (\beta/\tan\theta)\frac{d}{2}}{EI_b}V_{pd}a \right. \\ &\quad \left. + \frac{(1-\beta)a^2}{3EI_b}V_{pd}\right) \\ &= \frac{aV_{pd}}{2(1+\nu)I_b}\left(\frac{L'}{2} - \frac{\beta}{\tan\theta}\left(\frac{d}{2}\right) + \frac{(1-\beta)a}{3}\right)\end{aligned} \tag{A.12}$$

NOTES

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