

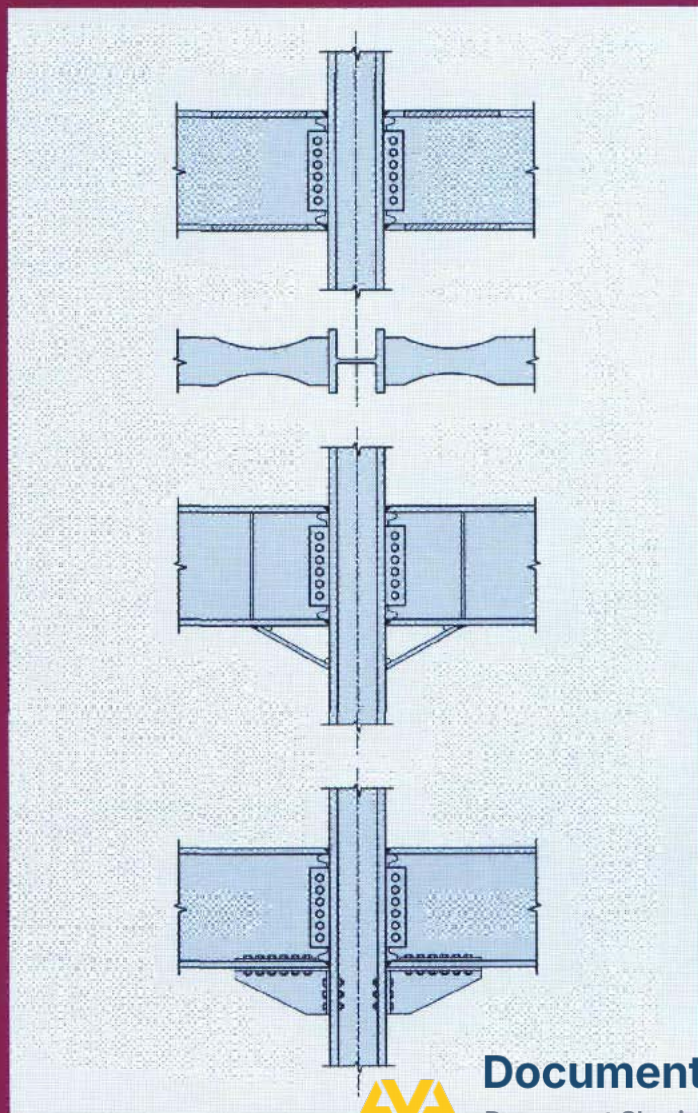


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# 12

## *Modification of Existing Welded Steel Moment Frame Connections for Seismic Resistance*

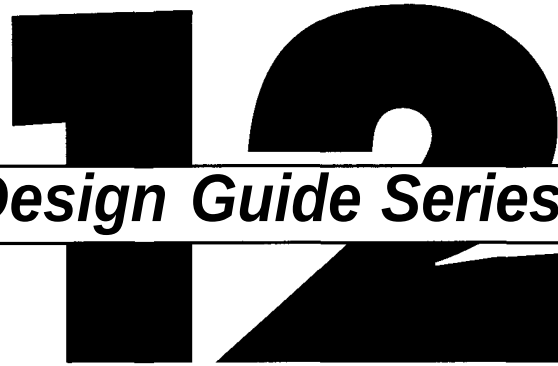


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# ***Modification of Existing Welded Steel Moment Frame Connections for Seismic Resistance***

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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<sup>1</sup> 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

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

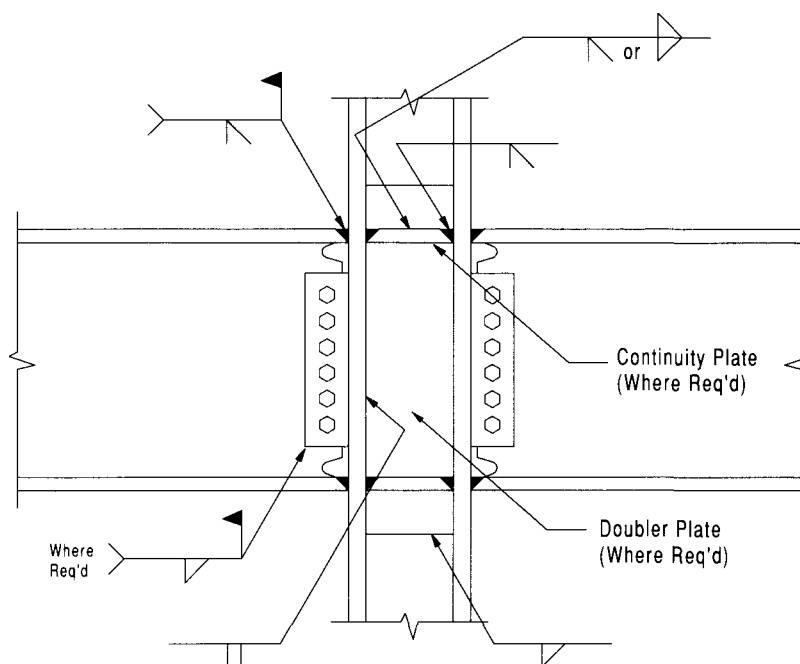


Figure 1.1 Prescribed Welded Beam-to-Column Moment Connection (Pre-Northridge)

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<sup>2</sup> 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.

## 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

<sup>2</sup>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).

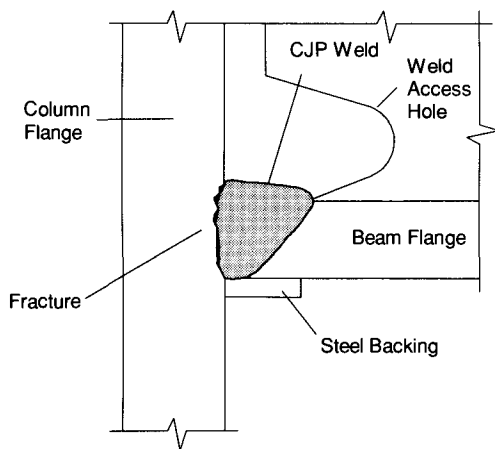


Figure 1.2 Typical Fracture Path

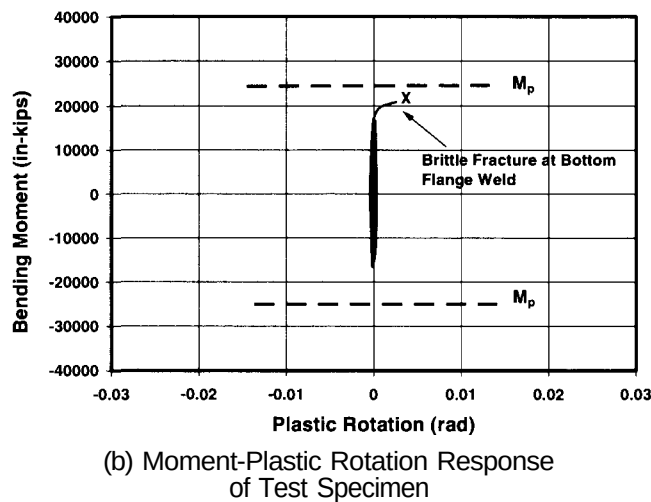
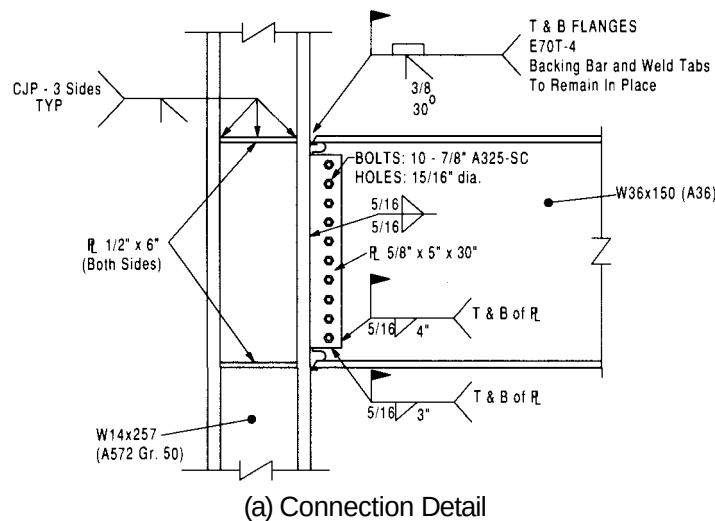


Figure 1.3 Laboratory Response of W36x150 Beam with pre-Northridge Connection

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).<sup>3</sup> 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 existing 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.

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<sup>3</sup>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

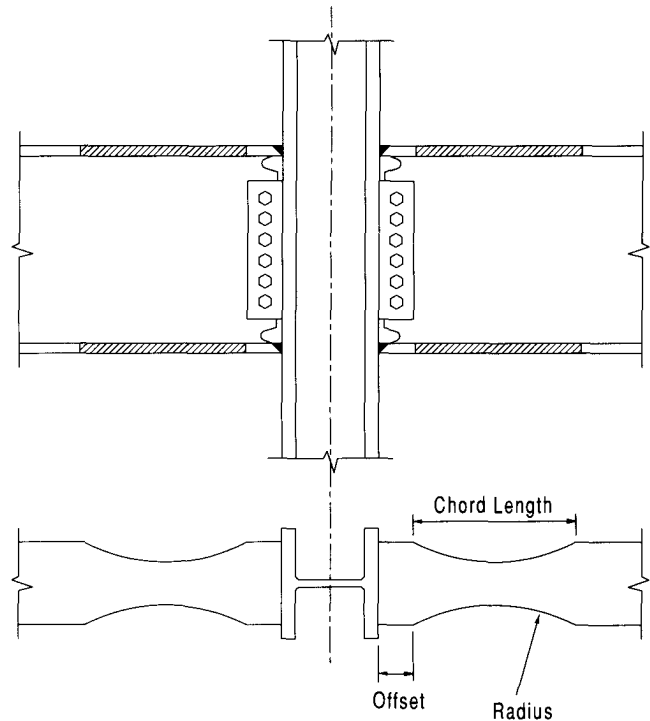


Figure 2.1 Reduced Beam Section (RBS)

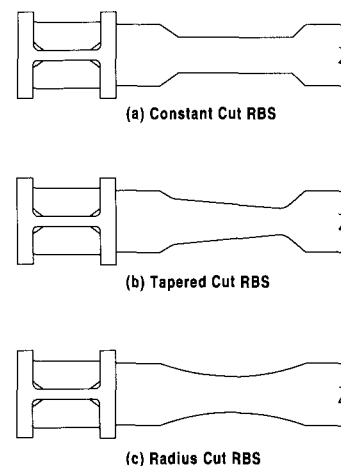


Figure 2.2 Typical Profiles of RBS Cutouts

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.

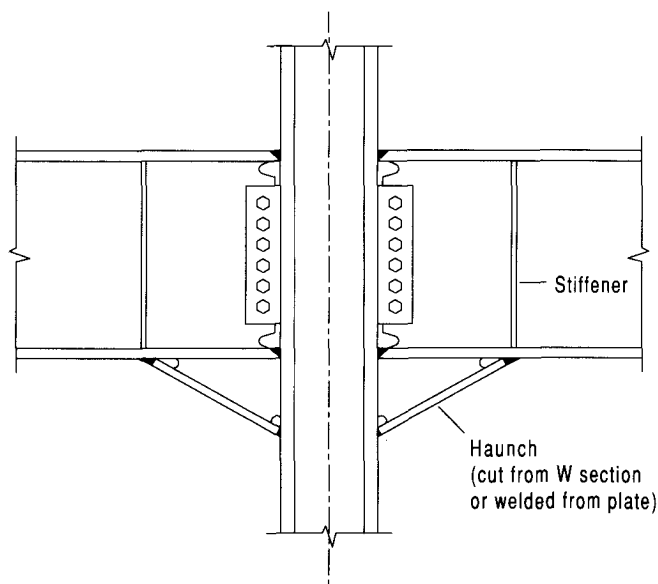


Figure 2.3 Welded Haunch

## 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.

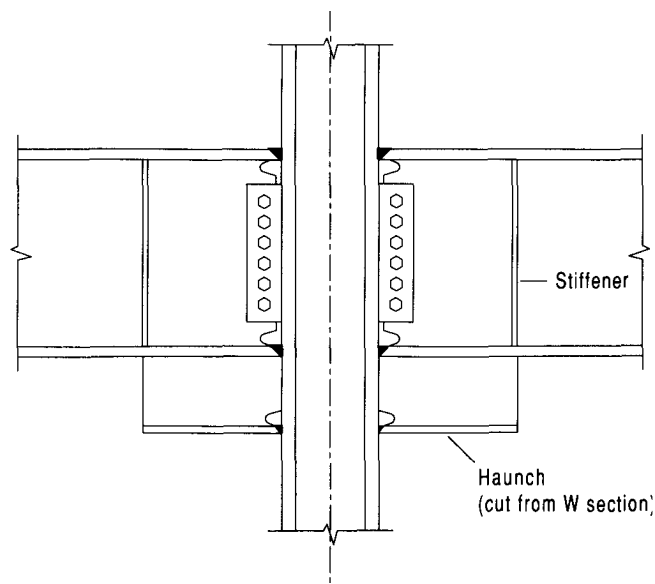


Figure 2.4 Straight Haunch

### 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.

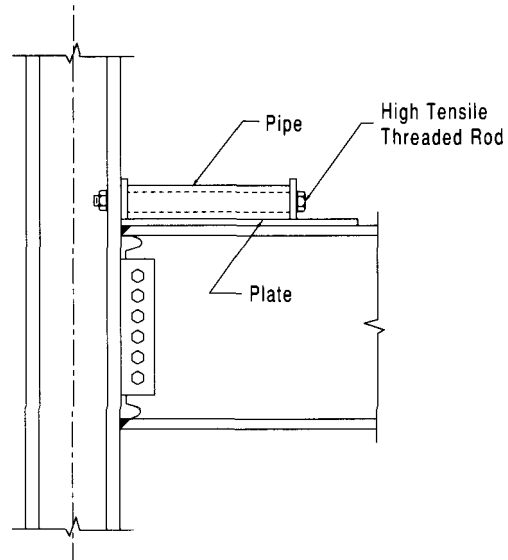


Figure 2.6 Pipe Bracket

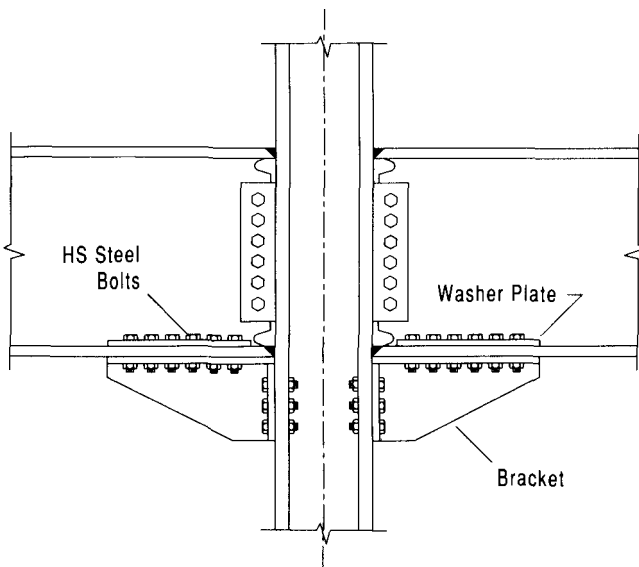


Figure 2.5 Bolted Bracket

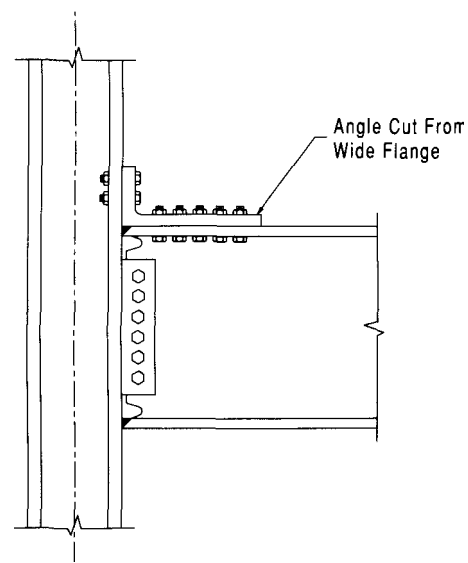


Figure 2.7 Angle Bracket

## 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         | RBSDetails and Other Flange Modifications                        | $\theta_p$ (%) | Comments                                                                             |
|------|----------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------------------------------------------------------------------------------------------|-----------------------------------------------------------|------------------------|------------------------------------------------------------------|----------------|--------------------------------------------------------------------------------------|
| [1]  | YC-1           | Built-up W shape<br>$d = 23.6"$ ,<br>$b_f = 11.8"$ ,<br>$t_f = 0.79"$ ,<br>$t_w = 0.47"$ ,<br>$L_b = 73"$<br>A36 steel<br>$F_{y-f} = 40$ ksi<br>$F_{u-f} = 66$ ksi<br>$F_{y-w} = 40$ ksi<br>$F_{u-w} = 65$ ksi | Built-up Box:<br>$19.7" \times 19.7" \times .79"$<br>$L_c = 87"$<br>A572 Gr. 50<br>$F_y = 56$ ksi<br>$F_u = 82$ ksi | FCAW<br>E70T-7<br>No weld tabs used                       | Bolted:<br>7-7/8" A325 | Tapered cut<br>$L_1 = 2"$<br>$L_{RBS} = 13.8"$<br>$FR = 20\%$    | 2.1            | Fracture of beam flange initiating at weld access hole                               |
| [1]  | YC-2           | "                                                                                                                                                                                                              | "                                                                                                                   | "                                                         | "                      | Tapered cut<br>$L_1 = 2"$<br>$L_{RBS} = 17.7"$<br>$FR = 25\%$    | 2.5            | Fracture of beam flange initiating at weld access hole                               |
| [1]  | PC-1           | "                                                                                                                                                                                                              | "                                                                                                                   | "                                                         | "                      | Tapered cut<br>$L_1 = 4.7"$<br>$L_{RBS} = 15.7"$<br>$FR = 34\%$  | 3.5            | Fracture of beam flange initiating at weld access hole                               |
| [1]  | PC-2           | "                                                                                                                                                                                                              | "                                                                                                                   | "                                                         | "                      | Tapered cut<br>$L_1 = 4.7"$<br>$L_{RBS} = 17.7"$<br>$FR = 42\%$  | 4.1            | Fracture of beam flange initiating at weld access hole                               |
| [1]  | PC-3           | "                                                                                                                                                                                                              | "                                                                                                                   | "                                                         | "                      | Tapered cut<br>$L_1 = 4.7"$<br>$L_{RBS} = 17.7"$<br>$FR = 42\%$  | 3.3            | Fracture of beam flange initiating at weld access hole                               |
| [2]  | DBT-1A-99-176  | W30×99<br>A572 Gr. 50<br>$L_b = 138"$<br>$F_{y-w} = 61.6$ ksi<br>$F_{u-w} = 82.8$ ksi                                                                                                                          | W14×176<br>A572 Gr. 50<br>$L_c = 168"$<br>$F_{y-w} = 55.6$ ksi<br>$F_{u-w} = 70.7$ ksi                              | FCAW<br>E70TG-K2;<br>backing bar removed at bottom flange | Bolted:<br>7-1" A325   | Tapered cut<br>$L_1 = 7.5"$<br>$L_{RBS} = 20.25"$<br>$FR = 45\%$ | 2.7            | no failure; test stopped due to limitations in test setup                            |
| [2]  | DBT-1B-99-176  | W30×99<br>A572 Gr. 50<br>$L_b = 138"$<br>$F_{y-w} = 51.5$ ksi<br>$F_{u-w} = 72.1$ ksi                                                                                                                          | W14×176<br>A572 Gr. 50<br>$L_c = 168"$<br>$F_{y-w} = 55.5$ ksi<br>$F_{u-w} = 71.8$ ksi                              | "                                                         | "                      | Tapered cut<br>$L_1 = 7.5"$<br>$L_{RBS} = 20.25"$<br>$FR = 45\%$ | 3.8            | no failure; test stopped due to limitations in test setup                            |
| [2]  | DBT-2A-150-257 | W36×150<br>A572 Gr. 50<br>$L_b = 138"$<br>$F_{y-w} = 60.2$ ksi<br>$F_{u-w} = 72.3$ ksi                                                                                                                         | W14×257<br>A572 Gr. 50<br>$L_c = 168"$<br>$F_{y-w} = 59.6$ ksi<br>$F_{u-w} = 75.2$ ksi                              | "                                                         | Bolted:<br>9-1" A325   | Tapered cut<br>$L_1 = 9"$<br>$L_{RBS} = 24"$<br>$FR = 45\%$      | 3.3            | Fracture of beam top flange near groove weld                                         |
| [2]  | DBT-2B-150-257 | W36×150<br>A572 Gr. 50<br>$L_b = 138"$<br>$F_{y-w} = 62.9$ ksi<br>$F_{u-w} = 83.1$ ksi                                                                                                                         | W14×257<br>A572 Gr. 50<br>$L_c = 168"$<br>$F_{y-w} = 64.5$ ksi<br>$F_{u-w} = 83.2$ ksi                              | "                                                         | "                      | Tapered cut<br>$L_1 = 9"$<br>$L_{RBS} = 24"$<br>$FR = 45\%$      | 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                                                                               | $\theta_p$ (%) | Comments                                  |
|-------|--------|--------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------|--------------------------------------------------------------------------------|--------------------------------------------------------------------------------------------------------------------------|----------------|-------------------------------------------|
| [3,4] | ARUP-1 | W36×150<br>A572 Gr. 50<br>$L_b = 132"$<br>$F_{y-f} = 55.5$ ksi<br>$F_{u-f} = 73$ ksi<br>$F_{y-w} = 62.5$ ksi<br>$F_{u-w} = 77$ ksi   | W14×426<br>A572 Gr. 50<br>$L_c = 136"$                                                                                                                         | FCAW<br>E70TG-K2<br>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<br>$L_1 = 9"$<br>$L_{RBS} = 24"$<br>$FR = 44\%$<br>top & bottom flanges reinforced with vertical ribs        | 3.3            | Flange fracture at minimum section of RBS |
| [3,4] | COH-1  | W27×178<br>A572 Gr. 50<br>$L_b = 132"$<br>$F_{y-f} = 44$ ksi<br>$F_{u-f} = 62$ ksi<br>$F_{y-w} = 46$ ksi<br>$F_{u-w} = 62$ ksi       | W14×455<br>A572 Gr. 50<br>$L_c = 136"$<br>$F_{y-f} = 55$ ksi<br>$F_{u-f} = 84$ ksi<br>$F_{y-w} = 54$ ksi<br>$F_{u-w} = 86$ ksi                                 | "                                                                                                              | "                                                                              | Tapered cut<br>$L_1 = 7"$<br>$L_{RBS} = 20"$<br>$FR = 38\%$<br>top & bottom flanges reinforced with vertical ribs        | 3.3            | "                                         |
| [3,4] | COH-2  | "                                                                                                                                    | "                                                                                                                                                              | "                                                                                                              | "                                                                              | "                                                                                                                        | 3.5            | "                                         |
| [3,4] | COH-3  | W33×152<br>A572 Gr. 50<br>$L_b = 132"$<br>$F_{y-f} = 57.6$ ksi<br>$F_{u-f} = 78.5$ ksi<br>$F_{y-w} = 62$ ksi<br>$F_{u-w} = 84.5$ ksi | W14×455<br>A572 Gr. 50<br>$L_c = 136"$<br>$F_{y-f} = 55$ ksi<br>$F_{u-f} = 84$ ksi<br>$F_{y-w} = 54$ ksi<br>$F_{u-w} = 86$ ksi<br>Beam connected to column web | "                                                                                                              | "                                                                              | Tapered cut<br>$L_1 = 9"$<br>$L_{RBS} = 26"$<br>$FR = 43\%$<br>top & bottom flanges reinforced with vertical side plates | 3.0            | "                                         |
| [3,4] | COH-4  | "                                                                                                                                    | "                                                                                                                                                              | "                                                                                                              | "                                                                              | "                                                                                                                        | 3.7            | "                                         |
| [3,4] | COH-5  | W33×152<br>A572 Gr. 50<br>$L_b = 132"$<br>$F_{y-f} = 62.8$ ksi<br>$F_{u-f} = 86$ ksi<br>$F_{y-w} = 69.1$ ksi<br>$F_{u-w} = 93.7$ ksi | "                                                                                                                                                              | "                                                                                                              | "                                                                              | "                                                                                                                        | 1.7            | "                                         |
| [5,6] | DB1    | W36×160<br>$L_b = 134"$<br>$F_{y-f} = 54.7$ ksi<br>$F_{u-f} = 75.6$ ksi<br>$F_{y-w} = 53.5$ ksi<br>$F_{u-w} = 79.2$ ksi              | W14×426<br>A572 Gr. 50<br>$L_c = 136"$                                                                                                                         | FCAW<br>E71T-8<br>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<br>$L_1 = 9"$<br>$L_{RBS} = 19.5"$<br>$FR = 40\%$                                                           | 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                                                                                                                                    | $\theta_p$ (%) | Comments                                                                                 |
|-------|-------|-------------------------------------------------------------------------------------------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------|----------------|---------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------|------------------------------------------------------------------------------------------|
| [5,6] | DB2   | W36×150<br>$L_b = 134"$<br>$F_{y-f} = 41.4$ ksi<br>$F_{u-f} = 58.7$ ksi<br>$F_{y-w} = 47.1$ ksi<br>$F_{u-w} = 61.8$ ksi | W14×426<br>A572 Gr. 50<br>$L_c = 136"$<br>$F_{y-f} = 50$ ksi<br>$F_{u-f} = 74.5$ ksi<br>$F_{y-w} = 50$ ksi<br>$F_{u-w} = 75$ ksi     | "              | "                               | Radius cut<br>$L_1 = 9"$<br>$L_{RBS} = 27"$<br>$FR = 40\%$                                                                                                                    | 2.8            | Testing stopped due to limitations of test setup                                         |
| [5,6] | DB3   | W36×170<br>$L_b = 134"$<br>$F_{y-f} = 58$ ksi<br>$F_{u-f} = 73$ ksi<br>$F_{y-w} = 58.5$ ksi<br>$F_{u-w} = 76.7$ ksi     | W14×426<br>A572 Gr. 50<br>$L_c = 136"$                                                                                               | "              | "                               | Radius cut<br>$L_1 = 9"$<br>$L_{RBS} = 27"$<br>$FR = 40\%$                                                                                                                    | 3.6            | "                                                                                        |
| [5,6] | DB4   | W36×194<br>$L_b = 134"$<br>$F_{y-f} = 38.5$ ksi<br>$F_{u-f} = 58.6$ ksi<br>$F_{y-w} = 43.6$ ksi<br>$F_{u-w} = 59.8$ ksi | W14×426<br>A572 Gr. 50<br>$L_c = 136"$<br>$F_{y-f} = 50$ ksi<br>$F_{u-f} = 74.5$ ksi<br>$F_{y-w} = 50$ ksi<br>$F_{u-w} = 75$ ksi     | "              | "                               | Radius cut<br>$L_1 = 9"$<br>$L_{RBS} = 27"$<br>$FR = 38\%$                                                                                                                    | 3.5            | "                                                                                        |
| [5,6] | DB5   | W30×148<br>$L_b = 134"$<br>$F_{y-f} = 46.6$ ksi<br>$F_{u-f} = 64.5$ ksi<br>$F_{y-w} = 48.5$ ksi<br>$F_{u-w} = 65.4$ ksi | W14×257<br>A572 Gr. 50<br>$L_c = 136"$<br>$F_{y-f} = 48.7$ ksi<br>$F_{u-f} = 69$ ksi<br>$F_{y-w} = 49.4$ ksi<br>$F_{u-w} = 66.2$ ksi | "              | "                               | Radius cut<br>$L_1 = 5"$<br>$L_{RBS} = 25"$<br>$FR = 38\%$                                                                                                                    | 3.8            | Testing stopped due to limitations of test setup; significant column panel zone yielding |
| [7]   | DB1   | W36×135<br>A36 Steel<br>$L_b = 134.5"$                                                                                  | W14×257<br>with 1-5/16" thk. cover plates (cover plates welded across flanges of W14×257 to from box)<br>A572 Gr. 50<br>$L_c = 132"$ | Not Available  | Not Available                   | Radius cut<br>$L_1 = 8"$<br>$L_{RBS} = 28"$<br>$FR = 40\%$                                                                                                                    | 2.8            | Testing stopped due to limitations of test setup                                         |
| [7]   | DB2   | W36×245<br>A36<br>$L_b = 134.5"$                                                                                        | W14×398<br>A572 Gr. 50<br>$L_c = 132"$                                                                                               | FCAW<br>E71T-8 | welded (heavy welded shear tab) | Radius cut<br>$L_1 = 11"$<br>$L_{RBS} = 25"$<br>$FR = 40\%$ ;<br>RBS provided in bottom flange only;<br>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                     | $\theta_p$ (%) | Comments                                                                                                                                         |
|------|-------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------|--------------------------------------------------------------------------------------------------------------|----------------------|----------------------------------------------------------------|----------------|--------------------------------------------------------------------------------------------------------------------------------------------------|
| [8]  | S-1   | W530×82<br>(Canadian Designation)<br>$d = 20.8"$ ,<br>$b_f = 8.2"$ ,<br>$t_f = 0.52"$ ,<br>$t_w = 0.37"$<br>wt. = 54 lb/ft.<br>$L_b = 142"$<br>CSA G40.41-350W steel<br>$F_{y-f} = 52.4$ ksi<br>$F_{u-f} = 76.6$ ksi<br>$F_{y-w} = 57.5$ ksi<br>$F_{u-w} = 81$ ksi | W14×120<br>A572 Gr. 50<br>$L_c = 120"$ | FCAW<br>E71T-8<br>backing bar left in place w/ seal weld at top flange; backing bar removed at bottom flange | Bolted:<br>5-1" A325 | Radius cut<br>$L_1 = 4.7"$<br>$L_{RBS} = 15.7"$<br>$FR = 55\%$ | 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 | $\theta_p$ (%) | 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 Lollo 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)  
 $\theta_p$  = 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.
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- [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.
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- [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   |                                                                                                                                                                          |                          | $\theta_p$<br>(%) | Comments                                                                                                                                                                               |
|------|----------|----------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------|--------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------|--------------------------|-------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
|      |          |                                                                                                                                        |                                        | Top flange               | Bottom flange                                                                                                                                                            | Beam Web                 |                   |                                                                                                                                                                                        |
| [1]  | UCB-RN2  | W36×150<br>A572 Gr. 50<br>$L_b = 135"$<br>$F_{y-f} = 60.6$ ksi<br>$F_{u-f} = 68.8$ ksi<br>$F_{y-w} = 60.1$ ksi<br>$F_{u-w} = 69.7$ ksi | W14×257<br>A572 Gr. 50<br>$L_c = 138"$ | pre-Northridge condition | Welded haunch<br>W18×143<br>A572 Gr. 50<br>$a = 0.5d_b$<br>$b = 0.27d_b$<br>$\theta = 28^\circ$ ;<br>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                                                                                                                              |                                                                                                                                                                          |                                                | $\theta_p$<br>(%) | Comments                                                                               |
|------|----------|--------------------------------------------------------------------------------------------------------------------------------|----------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------------------------------------------------|-------------------|----------------------------------------------------------------------------------------|
|      |          |                                                                                                                                |                                        | Top flange                                                                                                                                          | Bottom flange                                                                                                                                                            | Beam Web                                       |                   |                                                                                        |
| [1]  | UTA-1R   | W36×150<br>A36<br>$L_b = 135"$<br>$F_{y-f} = 42.3$ ksi<br>$F_{u-f} = 61.1$ ksi<br>$F_{y-w} = 47.7$ ksi<br>$F_{u-w} = 63.4$ ksi | W14×257<br>A572 Gr. 50<br>$L_c = 136"$ | replacing lower one-third of E70T-4 weld by E71T-8 weld; no steel backing                                                                           | Welded haunch<br>W18×143<br>A572 Gr. 50<br>$a = 0.5d_b$<br>$b = 0.31d_b$<br>$\theta = 31^\circ$ ;<br>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<br>A36<br>$L_b = 134"$<br>$F_{y-f} = 42.3$ ksi<br>$F_{u-f} = 61.1$ ksi<br>$F_{y-w} = 47.7$ ksi<br>$F_{u-w} = 63.4$ ksi | W14×257<br>A572 Gr. 50<br>$L_c = 136"$ | 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<br>A36<br>$L_b = 134"$<br>$F_{y-f} = 42.3$ ksi<br>$F_{u-f} = 61.1$ ksi<br>$F_{y-w} = 47.7$ ksi<br>$F_{u-w} = 63.4$ ksi | W14×257<br>A572 Gr. 50<br>$L_c = 136"$ | Welded haunch<br>W18×143<br>A572 Gr. 50<br>$a = 0.5d_b$<br>$b = 0.31d_b$<br>$\theta = 31^\circ$ ;<br>E70T-4 flange weld with defect left in place   | Welded haunch<br>W18×143<br>A572 Gr. 50<br>$a = 0.5d_b$<br>$b = 0.31d_b$<br>$\theta = 31^\circ$ ;<br>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<br>A36<br>$L_b = 134"$<br>$F_{y-f} = 48.6$ ksi<br>$F_{u-f} = 70.8$ ksi<br>$F_{y-w} = 57.4$ ksi<br>$F_{u-w} = 72.9$ ksi  | W14×176<br>A572 Gr. 50<br>$L_c = 136"$ | Welded haunch<br>W30×148<br>A572 Gr. 50<br>$a = 0.6d_b$<br>$b = 0.24d_b$<br>$\theta = 22^\circ$ ;<br>fractured beam flange disconnected from column | Welded haunch<br>W30×148<br>A572 Gr. 50<br>$a = 0.6d_b$<br>$b = 0.47d_b$<br>$\theta = 38^\circ$ ;<br>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**

| Ref. | Specimen | Beam                                                                                                                          | Column                                 | Rehabilitation Details                                                                                                                             |                                                                                                                                                   |                                                | $\theta_p$<br>(%) | Comments                                                                                                                                                                 |
|------|----------|-------------------------------------------------------------------------------------------------------------------------------|----------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------------------------------------------------------------------------------------------------------------------------|------------------------------------------------|-------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
|      |          |                                                                                                                               |                                        | Top flange                                                                                                                                         | Bottom flange                                                                                                                                     | Beam Web                                       |                   |                                                                                                                                                                          |
| [1]  | RFS-RN3  | W30×99<br>A36<br>$L_b = 134"$<br>$F_{y-f} = 47.2$ ksi<br>$F_{u-f} = 70.4$ ksi<br>$F_{y-w} = 53.4$ ksi<br>$F_{u-w} = 72.1$ ksi | W14×176<br>A572 Gr. 50<br>$L_c = 136"$ | Welded haunch<br>W24×103<br>A572 Gr. 50<br>$a = 0.6d_b$<br>$b = 0.24d_b$<br>$\theta = 22^\circ$ ;<br>E71T-8 flange weld with steel backing removed | Welded haunch<br>W14×82<br>A572 Gr. 50<br>$a = 0.6d_b$<br>$b = 0.41d_b$<br>$\theta = 34^\circ$ ;<br>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<br>A36<br>$L_b = 134"$<br>$F_{y-f} = 46.5$ ksi<br>$F_{u-f} = 67.7$ ksi<br>$F_{y-w} = 57.1$ ksi<br>$F_{u-w} = 72.5$ ksi | W14×176<br>A572 Gr. 50<br>$L_c = 136"$ | Pre-Northridge condition                                                                                                                           | Welded haunch<br>W21×93<br>A36<br>$a = 0.6d_b$<br>$b = 0.36d_b$<br>$\theta = 31^\circ$ ;<br>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<br>A36<br>$L_b = 134"$<br>$F_{y-f} = 46.5$ ksi<br>$F_{u-f} = 67.7$ ksi<br>$F_{y-w} = 57.1$ ksi<br>$F_{u-w} = 72.5$ ksi | W14×176<br>A572 Gr. 50<br>$L_c = 136"$ | E71T-8 weld between new flange plate and column flange; steel backing removed                                                                      | Welded haunch<br>W21×93<br>A36<br>$a = 0.6d_b$<br>$b = 0.36d_b$<br>$\theta = 31^\circ$ ;<br>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<br>A36<br>$L_b = 134"$<br>$F_{y-f} = 46.5$ ksi<br>$F_{u-f} = 67.7$ ksi<br>$F_{y-w} = 57.1$ ksi<br>$F_{u-w} = 72.5$ ksi | W14×176<br>A572 Gr. 50<br>$L_c = 136"$ | E70T-4 welded joint reinforced by two 3/4" thick rib plates underneath beam flange                                                                 | Welded haunch<br>W21×93<br>A36<br>$a = 0.6d_b$<br>$b = 0.36d_b$<br>$\theta = 31^\circ$ ;<br>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; $\theta_p$ 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                                                             |                                                                                                                                                                                                     |                                                                                                    | $\theta_p$<br>(%) | Comments                                                                                                                                                                                                                                  |
|------|----------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------|------------------------------------------------------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------|-------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
|      |                |                                                                                                                                                                                                                                                    |                                        | Top flange                                                                         | Bottom flange                                                                                                                                                                                       | Beam Web                                                                                           |                   |                                                                                                                                                                                                                                           |
| [2]  | UCSD-5R        | W30×99<br>A36<br>$L_b = 134"$<br>$F_{y-f} = 46.5$ ksi<br>$F_{u-f} = 67.7$ ksi<br>$F_{y-w} = 57.1$ ksi<br>$F_{u-w} = 72.5$ ksi                                                                                                                      | W14×176<br>A572 Gr. 50<br>$L_c = 136"$ | E71T-8 weld<br>with steel<br>backing<br>removed                                    | Welded<br>haunch<br>W21×93<br>A36<br>$a = 0.6d_b$<br>$b = 0.36d_b$<br>$\theta = 31^\circ$ ;<br>fractured beam<br>flange<br>disconnected<br>from column                                              | Groove weld<br>between beam<br>web and<br>column flange                                            | 1.4               | Low-cycle fatigue<br>fracture of beam<br>bottom flange<br>outside of haunch<br>due to local<br>buckling in four<br>dynamic test runs;<br>excellent energy<br>dissipation; $\theta_p$<br>limited by the<br>imposed maximum<br>displacement |
| [3]  | SFCCC-8        | W18×86 A36<br>$L_b = 120"$<br>$F_{y-f} = 50$ ksi<br>$F_{u-f} = 72$ ksi<br>$F_{y-w} = 51.5$ ksi<br>$F_{u-w} = 72.5$ ksi<br><br>W16×89<br>A572 Gr.50<br>$F_{y-f} = 47.7$ ksi<br>$F_{u-f} = 66.5$ ksi<br>$F_{y-w} = 47.5$ ksi<br>$F_{u-w} = 66.9$ ksi | W24×279<br>A572 Gr. 50<br>$L_c = 150"$ | Top flange<br>connected to<br>the column<br>through a 1-in<br>thick cover<br>plate | Welded built-<br>up haunch<br>1-in thick<br>flange, 3/4-in<br>thick web<br>$a = 1.0d_b$<br>$b = 0.67d_b$<br>$\theta = 34^\circ$ ;<br>beam flange<br>E71T-8 weld<br>with steel<br>backing<br>removed | Bolted to shear<br>plate                                                                           | 3.7<br>3.8        | Low-cycle fatigue<br>fracture of beam<br>flange outside of<br>haunch due to local<br>buckling                                                                                                                                             |
| [4]  | UTA<br>(NSF-4) | W36×150<br>A36<br>$L_b = 134"$<br>$F_{y-f} = 42.6$ ksi<br>$F_{u-f} = 61.7$ ksi<br>$F_{y-w} = 46.5$ ksi<br>$F_{u-w} = 63.0$ ksi                                                                                                                     | W14×455<br>A572 Gr. 50<br>$L_c = 136"$ | Pre-Northridge<br>connection<br>with steel<br>backing<br>removed                   | Welded<br>haunch<br>W18×143<br>A572 Gr. 50<br>$a = 0.56d_b$<br>$b = 0.56d_b$<br>$\theta = 45^\circ$ ;<br>beam flange<br>E70T-4 weld<br>with steel<br>backing<br>removed                             | Bolted to shear<br>plate;<br>supplemental<br>beam web<br>weld along<br>three sides of<br>shear tab | 1.5               | Weld fracture at top<br>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

$\theta$  = angle of sloped haunch

$\theta_p$  = 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;  $\theta_p$  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 subassembly 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                       |                                              | $\theta_p$<br>(%) | Comments                                   |
|---------|-------------------------------------------|-------------------------------------------|---------------------------------------------|---------------|----------------------------------------------|----------------------------------------------|-------------------|--------------------------------------------|
|         |                                           |                                           | Top flange                                  | Bottom flange | Top flange                                   | Bottom flange                                |                   |                                            |
| LK-HH-1 | W16×40<br>$F_y = 52.3$ ksi<br>$L = 120"$  | W16×40<br>$F_y = 52.3$ ksi<br>$L = 120"$  | Not welded                                  | Not welded    | Bolted haunch<br>no washer plate             | Bolted haunch<br>no washer plate             | 4.3<br>(4.6)      | Flange net<br>area fracture                |
| LK-WH-1 | W16×40<br>$F_y = 52.3$ ksi<br>$L = 120"$  | W12×65<br>$F_y = 53.1$ ksi<br>$L = 144"$  | E7018 weld,<br>backing bar<br>left in place | Not welded    | N/A                                          | Bolted haunch<br>washer plate                | >6.6<br>(>7.0)    | No failure                                 |
| LK-HH-2 | W36×150<br>$F_y = 38.4$ ksi<br>$L = 180"$ | W14×426<br>$F_y = 54.3$ ksi<br>$L = 144"$ | Not welded                                  | Not welded    | Bolted haunch<br>no plate                    | Bolted haunch<br>washer plate                | 5.1<br>(5.4)      | Flange net<br>area fracture                |
| LK-WH-2 | W36×150<br>$F_y = 38.4$ ksi<br>$L = 180"$ | W14×426<br>$F_y = 54.3$ ksi<br>$L = 144"$ | E71T-1 weld,<br>backing bar<br>removed      | Not welded    | N/A                                          | Bolted haunch<br>washer plate                | 5.0<br>(5.3)      | Flange net<br>area fracture                |
| LK-HH-3 | W36×150<br>$F_y = 38.4$ ksi<br>$L = 180"$ | W14×426<br>$F_y = 54.3$ ksi<br>$L = 144"$ | Not welded                                  | Not welded    | Bolted haunch<br>washer plate<br>brass plate | Bolted haunch<br>washer plate<br>brass plate | 5.9<br>(6.3)      | Flange<br>buckling, gross<br>area fracture |
| LK-PP-1 | W16×40<br>$F_y = 52.3$ ksi<br>$L = 120"$  | W12×65<br>$F_y = 53.1$ ksi<br>$L = 144"$  | Not welded                                  | Not welded    | Bolted pipe<br>no washer plate               | Bolted pipe<br>no washer plate               | 5.2<br>(5.5)      | Flange net<br>area fracture                |
| LK-WP-1 | W16×40<br>$F_y = 52.3$ ksi<br>$L = 120"$  | W12×65<br>$F_y = 53.1$ ksi<br>$L = 144"$  | E7018 weld,<br>backing bar<br>left in place | Not welded    | N/A                                          | Bolted pipe<br>washer plate<br>brass plate   | 7.5<br>(8.0)      | No failure                                 |
| LK-WP-2 | W36×150<br>$F_y = 38.4$ ksi<br>$L = 180"$ | W14×426<br>$F_y = 54.3$ ksi<br>$L = 144"$ | E71T weld<br>no backing<br>bar              | Not welded    | N/A                                          | Bolted pipe<br>washer plate<br>brass plate   | 6.6<br>(7.0)      | Flange net<br>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 and W36x150, 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 pre-date the 1994 Northridge Earthquake. The FCAW process was used to make the CJP flange welds and E70T-4

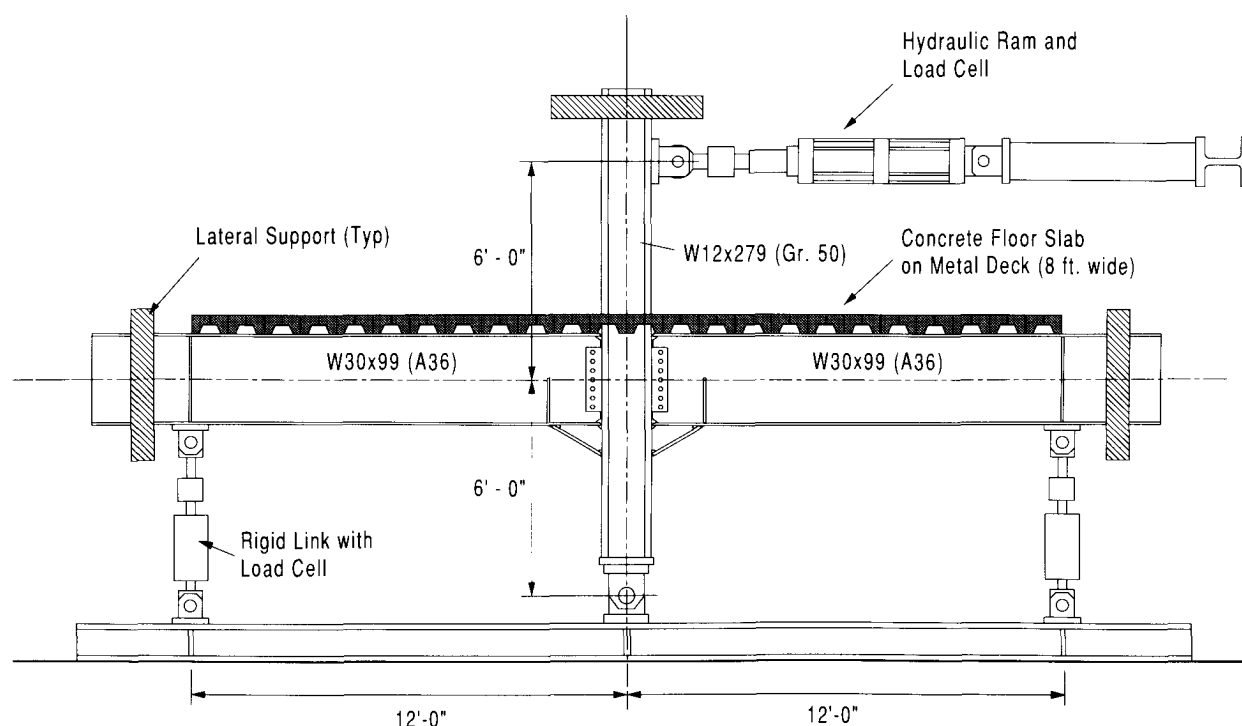


Figure 3.1 NIST/AISC Experimental Setup

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,  $\Delta_{CL}$ , was obtained by subtracting the elastic beam deflection from the total beam deflection. The plastic beam rotation,  $\theta_p$ , was determined from

$$\theta_p = \Delta_{CL}/L_{CL} \quad (3.1)$$

where

$\Delta_{CL}$  = 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  $\theta_p$  be the maximum plastic

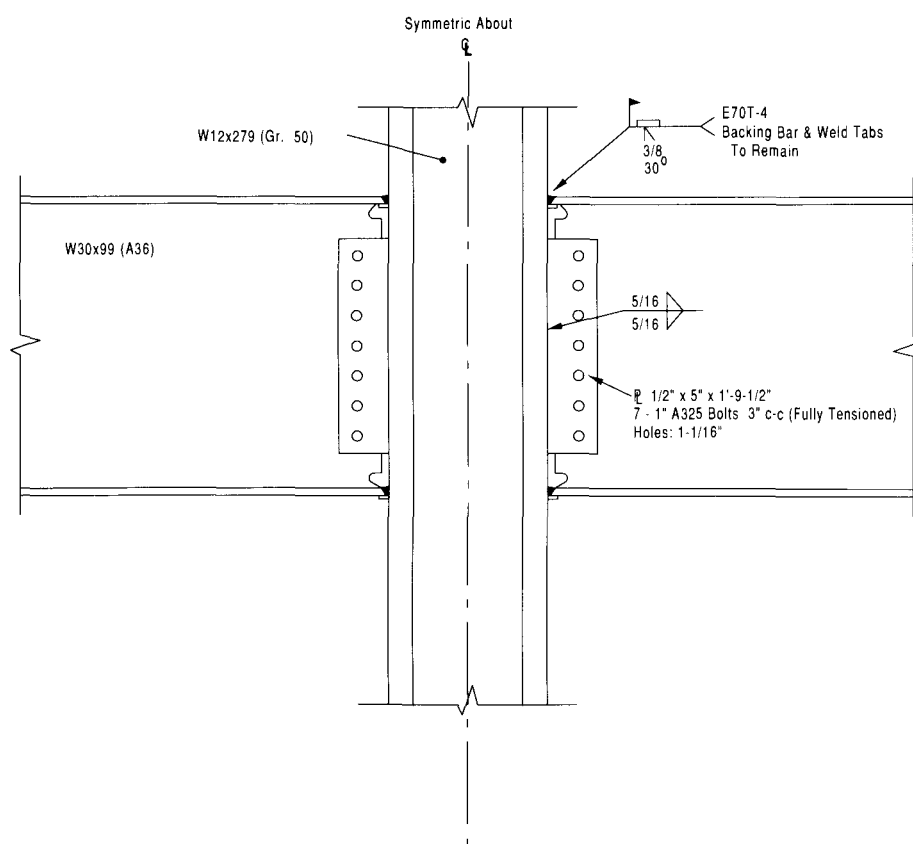


Figure 3.2 NIST/AISC Test Specimen Details

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

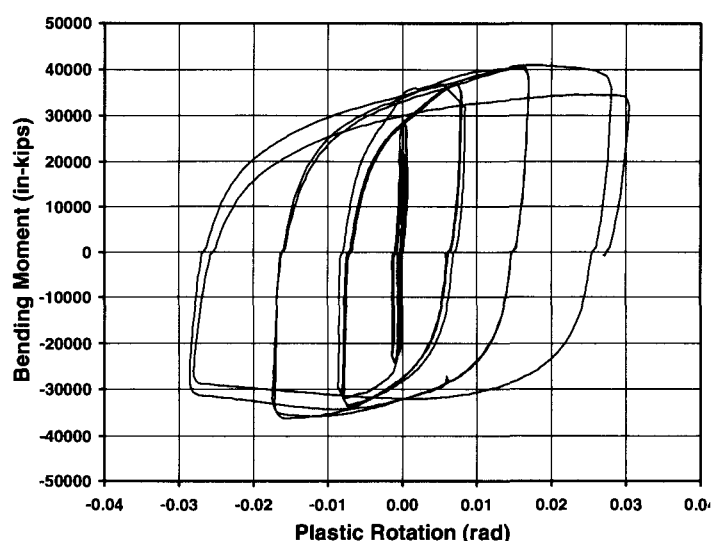


Figure 3.3 Moment-Plastic Rotation Response of a pre-Northridge Moment Connection with Welded Haunch Modification

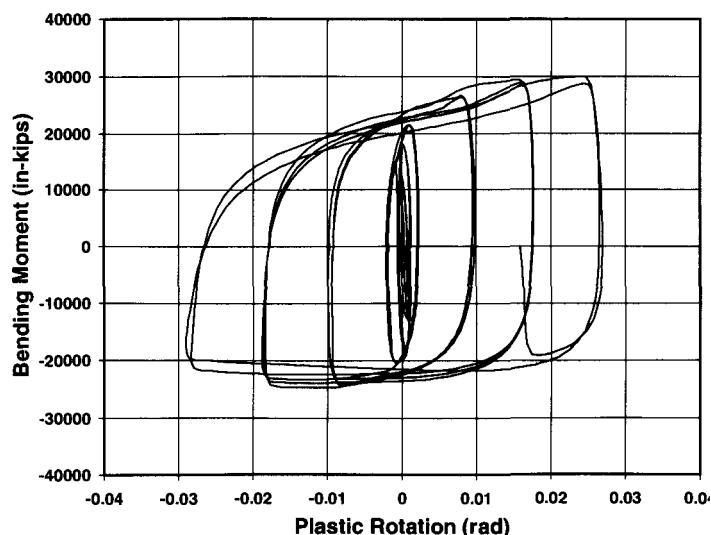


Figure 3.4 Moment-Plastic Rotation Response of a pre-Northridge Moment Connection with RBS Modification