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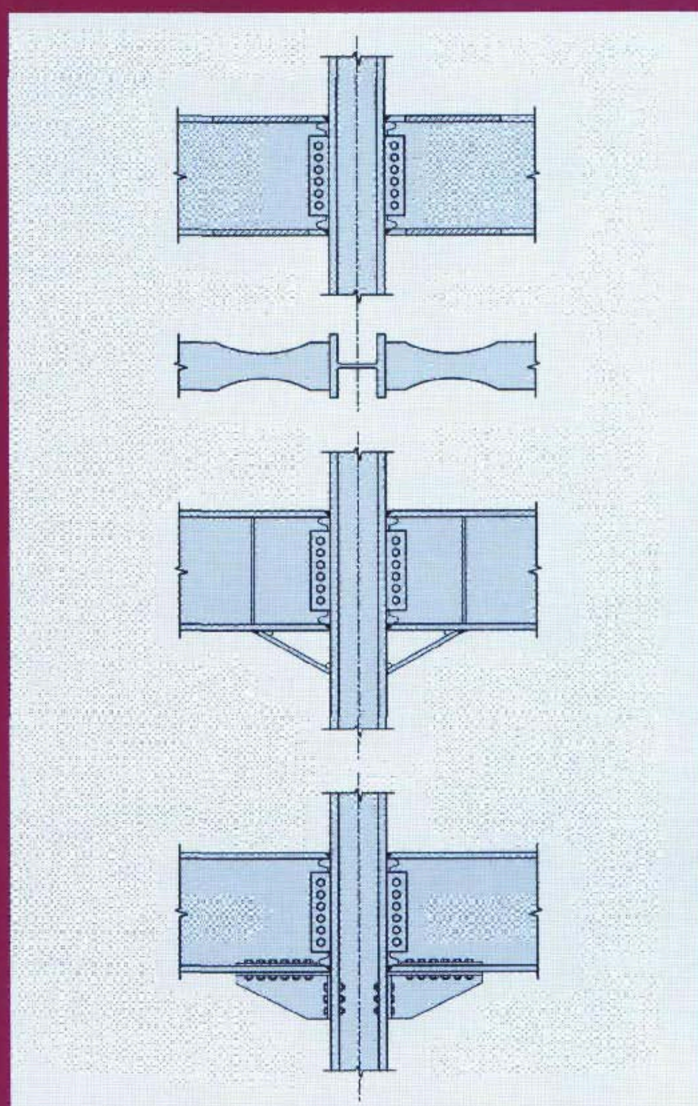
Steel Design Guide Series

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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Printed in the United States of America

Second Printing: October 2003

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Rev. 3/1/03

PREFACE

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

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

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

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

Chapter 1
INTRODUCTION

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

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

1.1 Background

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

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

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

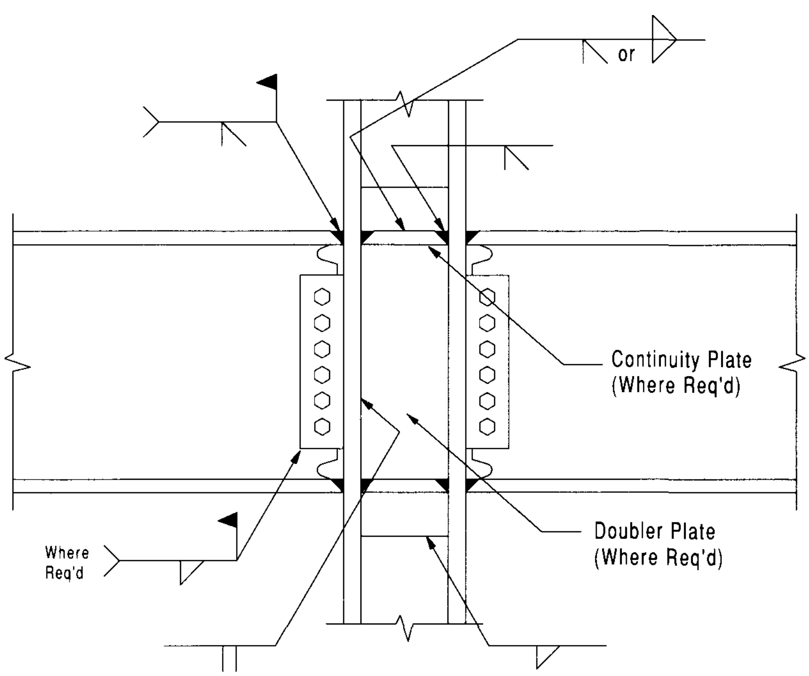


Figure 1.1 Prescribed Welded Beam-to-Column Moment Connection (Pre-Northridge)

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

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

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

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

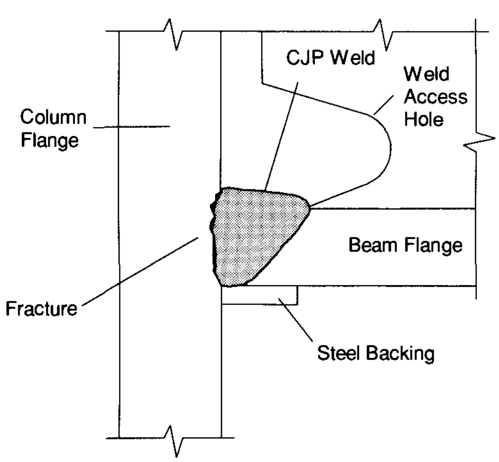


Figure 1.2 Typical Fracture Path

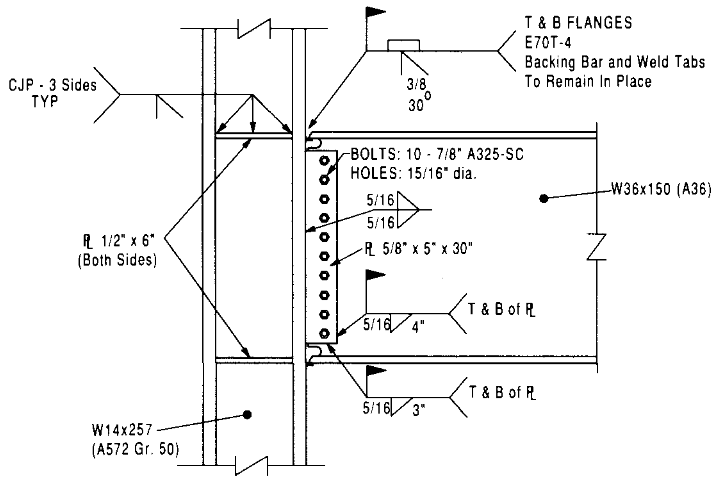
1.2 Factors Contributing to Connection Failures

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

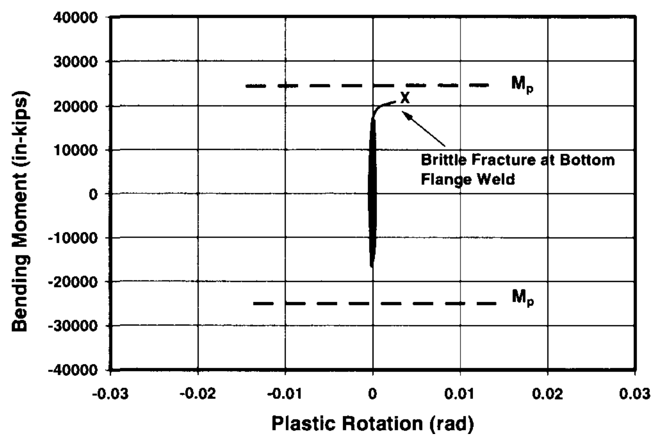
The self-shielded flux cored arc welding (FCAW) process is widely used for the CJP flange welds in WSMF construction. Electrodes in common use prior to the Northridge earthquake are not rated for notch toughness. Testing of welds samples removed from several buildings that experienced fractures in the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

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

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



(a) Connection Detail



(b) Moment-Plastic Rotation Response
of Test Specimen

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

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

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

1.3 Repair and Modification

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

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

1.4 Objective of Design Guide

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

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

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

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

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

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

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

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

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

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